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APPENDIX F

RIVERBANK ARMOR ELEVATION BACKUP

Appendix F
Riverbank Armor Height: Riparian Vegetation Approach

Objective

To prevent scour of the riverbank and mobilization of contaminated sediments.

Conceptual Approach

After riverbank soils are removed, banks will be revegetated using native species. Once plants or seeds are installed, there will be a period when plants are getting established (e.g., root development, ground cover) and the soil surface is prone to scour. During this period erosion control blankets will be utilized to prevent scour. The rate of plant establishment varies depending on the species, planting stock, plant maintenance, channel geometries, weather, soil, disease, and flooding, but generally will range from 1-3 years.

Riparian vegetation communities capable of providing scour resistance typically begin at some level up the bank slope. The specific elevation depends on the flood frequency and associated hydraulic characteristics (e.g., shear stress, scour resistance), plant species (e.g., tolerances to flooding), channel geometries, and bank soil types. Along a given riverbank slope, the transition from relatively unvegetated to vegetated varies for each river system but typically occurs at an elevation associated with flood events between the 1- to 3-year frequency. The specific flood frequency depends on the watershed, channel, and climatic characteristics and is called the bankfull event.

Assuming no other bank erosion process (e.g., slumping) is occurring and erosion blankets were installed below the elevation of the bankfull event (i.e., from the riverbed to the 1-yr event), they would initially provide short-term scour resistance. However, as the blankets deteriorated (2 - 4 years) they would begin to loose scour resistance and subsequently cause bank instability. Hence, to provide long-term scour resistance some type of armor is needed below bankfull elevation.

At river stages above bankfull, riparian vegetation becomes established within the life of the erosion blanket and then plant colonization provides the needed scour resistance over the long-term. The key design element for this stabilization approach depends on identification of the bankfull elevation.

Bankfull elevation

Field measurements taken during the aquatic habitat survey in July 2000 and HEC-RAS model results were utilized to estimate bankfull elevation. Channel measurements were collected from seven transects within the first reach of the 1.5-mile reach. The survey was conducted during low flow conditions (34 - 38 cfs) and bankfull elevations were measured at each transect using a stadia rod. The primary bankfull indicator used was a change in vegetation (e.g., from bare soil to grasses, herbs, and shrubs). From the low flow water surface, the average stage increase to bankfull was 2.2 ft. Because the vegetation transition was not always well defined, we recommend a more conservative estimate of 2.5 ft be used.

The HEC-RAS model showed the stage elevation of 40 cfs is approximately 971 ft. With the addition of the stage increase to bankfull (2.5 ft), bankfull elevation would be approximately **973.5 ft**. This would also be the top-of-the-bank armor elevation.

<u>Assumptions</u>

The armor height determination is based on the following assumptions:

(1) Critical transition is between bank armor and revegetation areas. Assumes that bioengineering methods (i.e., vegetated geogrids) have a higher scour resistance than straight revegetation methods (i.e., erosion blanket).

(2) Shear stress along the riverbank is greatest at the river bottom and decreases upslope. Average shear stress at the 10-yr event (design flood) ranges from 0.20 to 0.33 lbs/sf. (Based on HEC-RAS results; HC 1/01)

(3) Average channel velocity is greatest in the center of the channel near the surface and decreases towards the bed and the bank. Maximum channel velocity at this event ranges from 5.0 to 6.0 ft/s. (Based on HEC-RAS results; HC 1/01). Maximum channel velocity along the bank is estimated at 2.0 to 2.5 ft/s.

(4) The erosion blanket we are proposing is 100% coir with the following range of manufacturer specifications: max velocity 10 - 15 ft/s and shear stresses of 2.3 to 3.0 lbs/sf. These blankets are estimated to last 2 - 4 years with the assumption that herbaceous and woody vegetation will become established during decay and provide the needed soil protection afterwards

(5) Using a conservative factor of safety of 3.0 to account for pulses/localized maximums (Fischenich and Allen, 2000), manufacturer estimates for erosion blanket specifications, and HEC-RAS assumptions, the maximum bank shear stress and velocity are approximately 1.0 lbs/sf and 7.5 ft/s, respectively. These conditions are below specifications listed for the erosion blanket.

(6) Armor elevation is based solely on scour of revegetation areas. Additional analyses are also needed to fully assess and determine the bank armor elevation. These include a geotechnical analysis for slope stability, a more detailed assessment of the shear stress and velocity distribution along the riverbank, and an assessment of scour due to ice flows and woody debris.

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MEMORANDUM **DRAFT CONFIDENTIAL – FOIA EXEMPT**

DATE: March 9, 2001

TO: Hydraulics Focus Group

FROM: Shane Cherry

DRAFT
SUBJECT TO REVISION

RE: Top of Armor Elevation
J-7385-05

CC:

As part of the proposed clean up action, the riverbed and river banks will be restored and reinforced to protect against future scour and bank erosion. The previously recommended conceptual approach to bank armoring involves a combination of riprap armor, soil bioengineering, and vegetation. Both the effectiveness and the cost of this approach are sensitive to the elevation that defines the top of armor where riprap transitions to soil bioengineering or vegetation. Therefore the determination of this elevation is important and must be based on a compelling rationale.

Hart Crowser modeled the hydraulic conditions within the design reach using HEC-RAS. The hydraulic analysis provides a quantitative description of the flow velocities, flow depths, and boundary shear stress values throughout the design reach associated with floods ranging from the 1-year flood to the 100-year flood. Using the results of the hydraulic analysis, Hart Crowser determined an appropriate riprap size and gradation for placement within the riverbed and along the riverbanks. In parallel with the riprap analysis, and in cooperation with Woodlot Alternatives, Inc., Hart Crowser determined that bioengineering and vegetation would be adequate to protect against scour within the range of predicted shear stresses and velocities along the riverbanks down to the bank toe.

The establishment and persistence of thriving, living plants is essential to ensure the effectiveness of any soil bioengineering method. In addition to appropriately fitting the hydraulic conditions (velocity, depth, shear stress), the plants that form the bioengineering installations must be able to establish and persist under the anticipated hydrologic conditions (frequency and duration of inundation). On almost any river or stream one can observe a line of vegetation along the

Five Centerpointe Drive, Suite 240
Lake Oswego, Oregon 97035-8652
Fax 503.620.6918
Tel 503.620.7284

riverbank that corresponds to the edge of the active river channel. Such a line is readily observed along the Housatonic River within and adjacent to the design reach. Established vegetation persists above this line. Vegetation will rarely establish below this line, and when it does it is transitory and often dies or gets washed away. The vegetation line, along with other morphological features, is often used to identify the "bankfull channel", and it typically corresponds approximately to the 1.5-year flood water surface elevation.

Hart Crowser recommends that the top of armor elevation be placed at elevation 975 ft corresponding approximately to the 1.5-year flood water surface elevation. Bioengineering is not appropriate for installation below this elevation because plant material installed below this elevation may be subject to increased mortality. Plant mortality would compromise the integrity of bioengineering structures and reduce their effectiveness in protecting against scour and bank erosion.

APPENDIX G

SUMMARY TABLES OF PROPOSED RIVERBANK GRADES AND STABILIZATION METHOD

Proposed Riverbank Grades and Stabilization Method [Woodlot Alternatives 7/7/01]:

Station	West Bank						East Bank						Estimated Station Impacts	
	Bank Armor		Slope Above Bank Armor			Comments	Bank Armor		Slope Above Bank Armor			Comments	Excavation Increase	FSC Change
	Existing Grade (H:1V)	Proposed Grade (H:1V)	Existing Grade (H:1V)	Proposed Grade (H:1V)	Stabilization		Existing Grade (H:1V)	Proposed Grade (H:1V)	Existing Grade (H:1V)	Proposed Grade (H:1V)	Stabilization			
500+00	*1.6	**1.6**	*1.6; flat	**1.6; flat**	**Rock Armor**	Drainage feature present? Grade change occurs at 500+15 (armor to soil transition); decrease grade to 2.5:1 (limit of excavation increase will be needed).	NA		NA		Rock Armor			
500+50	1.6	**1.8**	*2.1	**2.5**	Revegetation	Decrease bank armor grade (Geotech). Increase in limit of excavation needed (3.1 ft).	NA		NA		Bioengineering		X	I
501+00	2.0	**2.0**	2; 4	**2.9**	Revegetation	Decrease revegetation grade (Construction).	NA		NA		Bioengineering		X	I
501+50	1.7	**1.8**	2; *7.8	**2.8**	Revegetation	Decrease bank armor grade (Geotech) and revegetation grade (Construction).	NA		NA		Bioengineering		X	I
502+00	2.9	**3.0**	2.5; 10	**3.0**	Revegetation	Decrease bank armor and revegetation grade (Construction).	NA		NA		Bioengineering	Undisturbed bank portion between end of bioengineering and Sta 502+50. Check tie in.	X	I
502+50	2.4	**2.4**	2.7; 11.1	**2.7; 11.1**	Revegetation	Composite slope.	1.3	**1.5**	*1.3; flat	**2.7**	Revegetation	Decrease bank armor grade (CENAE). Decrease revegetation grade (Construction/Geotech).		
503+00	2.0	**2.0**	*4	**3.6**	Revegetation	Increase revegetation grade (Construction).	0.8	**1.5**	0.9; 6.3	**2.0**	Revegetation	Decrease bank armor grade (CENAE/Geotech). Increase in limit of excavation needed (0.5 ft).		
503+50	2.4	**2.4**	2.3; 10.6	**2.5**	Revegetation		1.1	**1.5**	1.3; 8.0	**2.0; 8.0**	Revegetation	Decrease bank armor grade (CENAE/Geotech). Composite revegetation slope.		
504+10	1.5	**2.0**	2; flat	**2.5**	Revegetation	Decrease bank armor and revegetation grade (Geotech). Outfall present @ STA 504+00; Station moved downstream to be more representative.	NA	**NA**			None	Drainage Swale Outlet	X	I
504+50	1.9	**2.3**	1.8; 4	**2.3**	Revegetation.	Decrease bank armor and revegetation grade (Geotech).	*1.8	**1.8**	*2.9	**2.9**	Revegetation		X	I
505+00	1.2	**2.3**	2.3	**2.3**	Revegetation	Decrease bank armor grade (Geotech). Small pool gets partially filled.	1.8	**1.8**	0.8; 1.3; flat	**2.0**	Revegetation	Increase in limit of excavation needed (1.7 ft).	X	
505+50	1.8	**2.3**	1.9	**2.3**	Revegetation	Decrease bank armor and revegetation grade (Geotech). Increase in limit of excavation needed (2.0 ft).	1.7	**1.7**	1.5; 28.7; 2.0	**2.0; flat; 2.2**	Revegetation	Decrease revegetation grade (Restoration/Construction). Composite revegetation slope.	X	
506+00	1.0	**2.0**	1; 4	**2.8**	Revegetation	Decrease bank armor and revegetation grade (Geotech/Construction).	1.8	**1.8**	*2.0; 22.4	**2.1;22.4**	Revegetation	Composite revegetation slope.		D
506+50	1.5	**2**	1.5; flat	**2.4**	Revegetation	Decrease bank armor and revegetation grade (Geotech).	1.3	**1.5**	7.4; 2.4; flat; 1.8; 4.0	**7.4; 2.4; flat; 2.5**	Revegetation	Decrease bank armor grade (CENAE). Composite revegetation slope (Construction).		
507+00	2.1	**2.2**	*3.0	**3.0**	Revegetation	Revegetation ends at STA 507 + 10.	0.9	**1.5**	0.8; 4:1; flat; -10.0; 2; 13.8	**2.0; flat; 2.0; flat**	Revegetation	Decrease bank armor grade (CENAE). Composite revegetation slope.		
507+50 (outfall)	0.3	**NA**	0.5; *6.3	**NA**	Rock armor	Design completed by Hart Crowser, Inc. Fill proposed.	0.9	**1.5**	1.0; *3.4; -flat; 1.8; flat	**2.6; flat; 2.0; flat**	Revegetation	Decrease bank armor grade (CENAE). Composite revegetation slope.		D
508+00	0.8	**2.2**	2.5; flat	**2.2**	Rock armor	Decrease bank armor grade (Geotech). Increase in limit of excavation needed (1.4 ft). Entire riverbank slope is armor.	2.0	**2.0**	0.8; 8.0; *3; *flat; 2.4	**3.0; 7.8; flat; 2.4**	Revegetation	Top-of-bank armor reduced to 973.5 feet elevation (Restoration). Composite revegetation slope (Construction).	X	I
508+50	1.2	**2.2**	1.0	**2.2**	Rock armor	Decrease bank armor grade (Geotech). Increase in limit of excavation needed (6.0 ft). Entire riverbank slope is armor.	5.4	**5.4**	**4.5; 6.2; 3.9**	**4.5; 6.2; 3.9**	Revegetation	Top-of-bank armor reduced to 973.5 feet elevation (Restoration).	X	D

Station	West Bank						East Bank						Estimated Station Impacts	
	Bank Armor		Slope Above Bank Armor			Comments	Bank Armor		Slope Above Bank Armor			Comments	Excavation Increase	FSC Change
	Existing Grade (H:1V)	Proposed Grade (H:1V)	Existing Grade (H:1V)	Proposed Grade (H:1V)	Stabilization		Existing Grade (H:1V)	Proposed Grade (H:1V)	Existing Grade (H:1V)	Proposed Grade (H:1V)	Stabilization			
509+00	0.5	**2.2**	0.5; flat	**2.2**	Rock armor	Decrease bank armor grade (Geotech). Increase in limit of excavation needed (5.6 ft). Entire riverbank slope is armor.	4.5	**4.5**	4.3; -6.0; 4.8; 1.5	**4.3; flat; 6.6; 2.0**	Revegetation	Top-of-bank armor reduced to 973.5 feet elevation (Restoration). Composite revegetation slope (Construction/Geotech).	X	I
509+50	2.1	**2.2**	NA	**2.2**	Rock armor	Limit of excavation currently below 975 ft elevation. Increase in limit of excavation needed (9.3 ft). Riverbank all armor. Decrease grade (Geotech).	3.9	**3.9**	*3.0; flat; 1.0; 3.3; flat	**2.9; flat; 3.4**	Revegetation	Decrease revegetation grade (Construction/Restoration). Composite revegetation slope.		
510+00	2.3	**2.2**	NA	**2.2**	Rock armor	Limit of excavation currently at 975 ft elevation. Rock armor ends at STA 510 + 25. Increase in limit of excavation needed (4.9 ft).	2.3	**2.3**	2.3; 4.0; flat	**2.7; flat**	Revegetation	Decrease revegetation slope (Construction). Composite revegetation slope.	X	I
510+50	*2.2	**2.2**	2.5	**2.5**	Revegetation	Small portion for revegetation (~ 3 ft slope length) – consider boundary extension?	1.0	**2.0**	1.0; -8.0; 10.0; 4.0	**4.7**	Revegetation	Decrease bank armor grade (Geotech/Restoration). Change in revegetation slope (Construction).	X	I
511+00	1.6	**1.8**	*2.6; flat	**2.6; flat**	Revegetation	Composite revegetation slope.	4.3	**4.3**	7.7; *2.9	**7.7; 2.9**	Revegetation	Composite revegetation slope.		
511+50	1.8	**1.8**	1.8; 6.0	**2.5**	Revegetation	Decrease revegetation grade (Construction).	1.5	**2.0**	*3.0	**3.0**	Revegetation	Decrease bank armor grade (Geotech).		
512+00	2.0	**2.2**	2.0; *4.0	**2.9**	Revegetation	Decreased bank armor and revegetation grades (Construction/Restoration/Geotech).	0.4	**1.7**	0.4; *1.8; flat	**2.2**	Revegetation	Decrease bank armor grade (Geotech).	X	
512+50	1.8	**1.8**	*14.6; 3.4	**14.6; 3.4**	Revegetation	Composite revegetation slope.	1.2	**1.7**	1.3; 9.0	**2.5**	Revegetation	Decrease bank armor grade (Geotech).	X	
513+00	1.8	**2.2**	6.0; *flat	**6.1; flat**	Revegetation	Extra excavation to compensate fill at Sta 513+00 East Bank. Estimated so overall station X-area remained constant. Decreased bank armor and revegetation grades (Restoration/Construction).	0.8	**1.7**	0.8; 4.0	**1.8; 4.8**	Bioengineering (3 soil lifts)	Decrease bank armor grade (Geotech). Use rock swale for first 5.0 ft to tie in upstream end of soil lifts. All soil lifts from Sta 513+00 to 514+00 are approximately same grade (Restoration).	X	
513+50	*5.8	**5.8**	*5.4; *2.5; 8.3	**6.2; 3.3**	Revegetation	Extra excavation to compensate fill at Sta 513+50 East Bank. Estimated so overall station X-area remained constant. Composite revegetation slope (Construction/Restoration). Top-of-bank armor reduced to 973.5 feet elevation (Restoration).	0.8	**1.7**	1.0; 1.5; 4.0	**1.8; 2.8**	Bioengineering (3 soil lifts)	Decrease bank armor grade (Geotech).	X	
514+00	3.6	**3.6**	6.5; *3.4	**6.0; 3.4**	Revegetation	Top-of-bank armor reduced to 973.5 feet elevation (Restoration). Composite revegetation slope.	5.0; *1.0	**1.7**	1.6; *1.3	**1.8; 2.0**	Bioengineering (4 soil lifts)	Decrease bank armor grade (Geotech/Construction). Add one additional soil lift. Bioengineering ends at STA 513+95. Use rock swale to tie bioengineering into bank (513+95 to 514). Swale could also be used for flood drainage from overland flow. Increase in limit of excavation needed (3.2 ft).	X	

Notes:
(1) Bank slope lengths and grades based on R.F. Weston 2000 topography.
(2) Assumed that bank armor elevation (975 feet) remains constant in this reach, except where noted.
(3) Designations a, b, c, and d refer to subsections of a composite slope. "a" starts at top of bank armor and "d" is the last subsection near top of bank.
(4) "*" indicates that two slopes were lumped into one slope (slight grade changes between them).
(5) Bank armor elevation lowered to 973.5 ft in a few locations because bank slopes are relatively flat (<3:1), and these stations are located in the inside of a channel bend where sediment transport is expected to be depositional and velocities and shear stresses are expected to be relatively lower than the main channel. The objectives are to lower armor costs and increase the revegetation area.
(6) Design Comments: "CENAE Guideline" refers to recommendation by Don Wood (Corps) on 4/4/01 regarding maximum acceptable bank armor grades (i.e., 1.5:1). "Geotech" refers to maximum slope grade based on stability analysis conducted by Hart Crowser Inc. (L. Jen 4/4/01). "Restoration" refers to changes needed to meet restoration needs (e.g., armor grade transitions between stations, compensation needed to maintain flood storage capacity, or needs for bioengineering construction/design such as horizontal length of soil lift). "Construction" refers to the needs to reconstruct restoration slope efficiently (e.g., reducing the number of slope grades).
(7) FSC refers to Flood Storage Capacity (Estimated change: I = Increase, D = Decrease).
(8) Rock armor is proposed for the hard structure design from Sta 508+00 to 510+00. The existing bank armor will be extended above 975 ft elevation. Design slopes based on geotechnical assessment (i.e., Hart Crowser, Inc. slope stability analysis (L.Jen 4/12/01)). Objectives for restoration included maintaining a constant grade (i.e., 2.2:1) and a relative constant elevation (i.e., 978 to 977 ft) through these stations.

Proposed Riverbank Grades and Stabilization Method - Drainage Swale

(Reach 1; 1 1/2-Mile Reach; GE/Housatonic River Site; Pittsfield, MA)

DRAFT CONFIDENTIAL, FOIA Exempt [April 17, 2001, Woodlot Alternatives, Inc. (ws/kh)]

Station	Bank Armor						Comments	Existing Restoration Slope (above armor)			Morphology	Proposed Restoration Slope (above armor)			Slope Adjustment	Proposed Stabilization	Comments
	Existing				Proposed			Slope Leg	Grade (H:1V)	Slope Length (ft)		Bank Height (ft)	Slope Length (ft)	Proposed Grade (H:1V)			
	Toe Elev (ft)	Height (ft)	Horizontal Length (ft)	Avg. Grade (H:1V)	Slope Length (ft)	Proposed Grade (H:1V)											
North Bank																	
0+50	970	5	12.9	2.6	13.9	**2.6**	Fill needed. Regrade slope.	a	1.7	7.9	Run	3.8	8.4	**2.0**	Excavation needed. Composite slope (small berm present on top of bank).	Revegetation	Approximate equal fill and excavation.
								b	-3.1	4.3		2.4	4.4	**-3.1**			
1+00	972	2.9	7.7	*2.7	8.8	**2.7**		a	3.7	9.90	Run	2.5	9.5	**3.8**		Revegetation	
1+50	971	3.3	13.1	4.0	14.2	**3.4**		a	*3.0	10.00	Run	3.1	10	**3.1**	Excavation needed. Regrade slope to 3:1	Revegetation	
South Bank																	
0+50	970	5	11.3	*2.3	12.5	**2.3**	Excavation and fill needed.	a	4.5	2.5	Run	4.8	13	**2.5**	Excavation needed. Regrade slope to 2.5:1	Revegetation	
								b	1.1	6.2							
								c	15.8	6.0							
1+00	972	3.1	6	1.9	5.9	**1.9**	Fill needed.	a	2.1	17.3	Run	7.7	17.3	**2.1**		Revegetation	
1+50	971	4	8.3	2.1	8.8	**2.1**		a	2.5	18.5	Run	9.3	20.6	**2.0**		Revegetation	
								b	flat	2.5							

Notes:
(1) Bank slope lengths and grades based on R.F. Weston 2000 topography.
(2) Assumed that bank armor elevation (975 feet) remains constant in this reach.
(3) Designations a, b, c, and d refer to subsections of a composite slope. "a" starts at top of bank armor and "d" is the last subsection near top of bank
(4) "*" indicates that two or more slopes were lumped into one slope (slight grade changes between them).

APPENDIX H

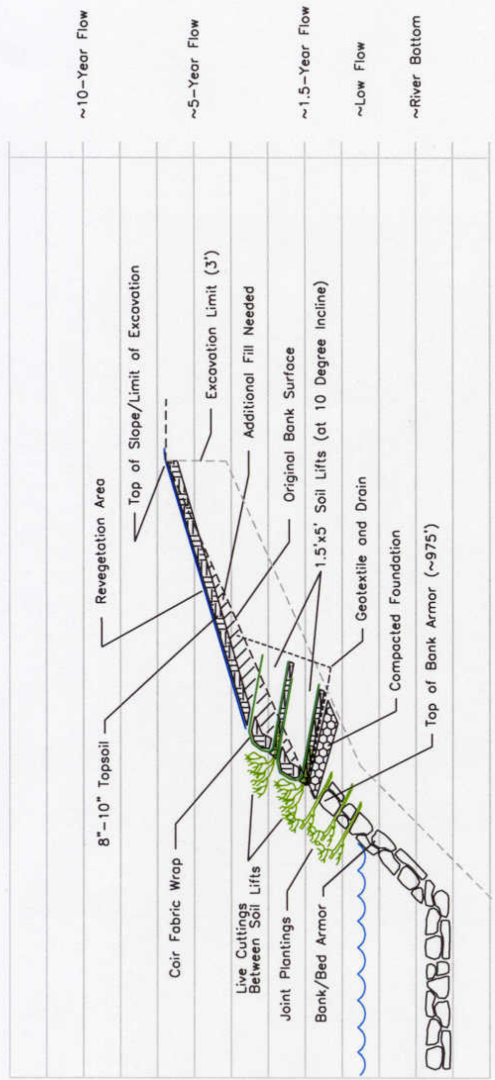
DESIGN CONSIDERATIONS OF BIOENGINEERING METHODS

Relative comparison of design factors for each bioengineering alternative [Woodlot Alternatives, Inc 4/2/01]:

Factor	Vegetated Geogrid[1]	Rock Wall Terraces[2]	Live Fascines	Brush Layers	Brush Mattress
Relative Cost[3]	$415/lf	$415/lf	$311/lf	$422/lf	$360/lf
Maximum Slope Applicability	1:1	2.25:1	2.5:1	1.5:1	2:1
Additional Cut or Fill	Fill	Cut	No Change	No Change	No Change
Initial Slope Stability	High	High	Moderate	Moderate	Moderate
Long-term Stability	High	High	High	High	High
Complexity for Construction	High	High	Low	High	Moderate
Labor/Time	High	High	Moderate	Moderate	Moderate
Training Needed	High	High	Moderate	Moderate	Moderate
Maintenance	Low	Low	Low	Low	Low
Ease of Replanting	Moderate	Easy	Easy	Moderate	Moderate
Construction Schedule Flexibility	High	High	High	Low	Low
Plant Contract Complexity	Moderate	Moderate	Moderate	Moderate	Moderate
Installation Period[4]	Year-round	Year-round	Nov-May	Nov-May	Nov-May
Ease of Merging with Other Structures	Varies	Complex	Easy	Varies	Easy

Note:
1. Relative cost estimate for vegetated geogrid is based on using two vegetated geogrids (i.e., soil lifts). If three geogrids are used, the cost increases to approximately $500/lf.
2. Relative cost estimate for rock wall terraces is based on using one 3-foot wall. If two 3-foot walls are used, the cost increases to approximately $620/lf.
3. Relative cost estimates are based on the best available information and include costs of topsoil, plants, fill, materials, equipment, transportation, and labor. Costs presented are based on a typical bank configuration for a 50-foot section of bank (one side only), with a slope length of 20 feet, a slope of 2.5 H: 1V, and a total area of 1,000 square feet.
4. Installation period for geogrids and rock terraces assumes cuttings would be used Nov-April and containerized plants would be used April -Nov. Total plant costs include propagation, storage, handling, and installation, and are approximately the same for cuttings and containerized plants.

Vegetated Geogrid
(Typical for 20' slope @ 2.5:1)



Note:

The minimum number of soil lifts in bioengineering applications will be two. Up to five 1.5-foot soil lifts may be needed, depending on bank height, slope length, and slope angle for the particular river bank section.

For example, for a 20-foot slope length:

3-5 Lifts	2-3 Lifts		Number of Soil Lifts
1:1	2:1	3:1	Original Slope Angle

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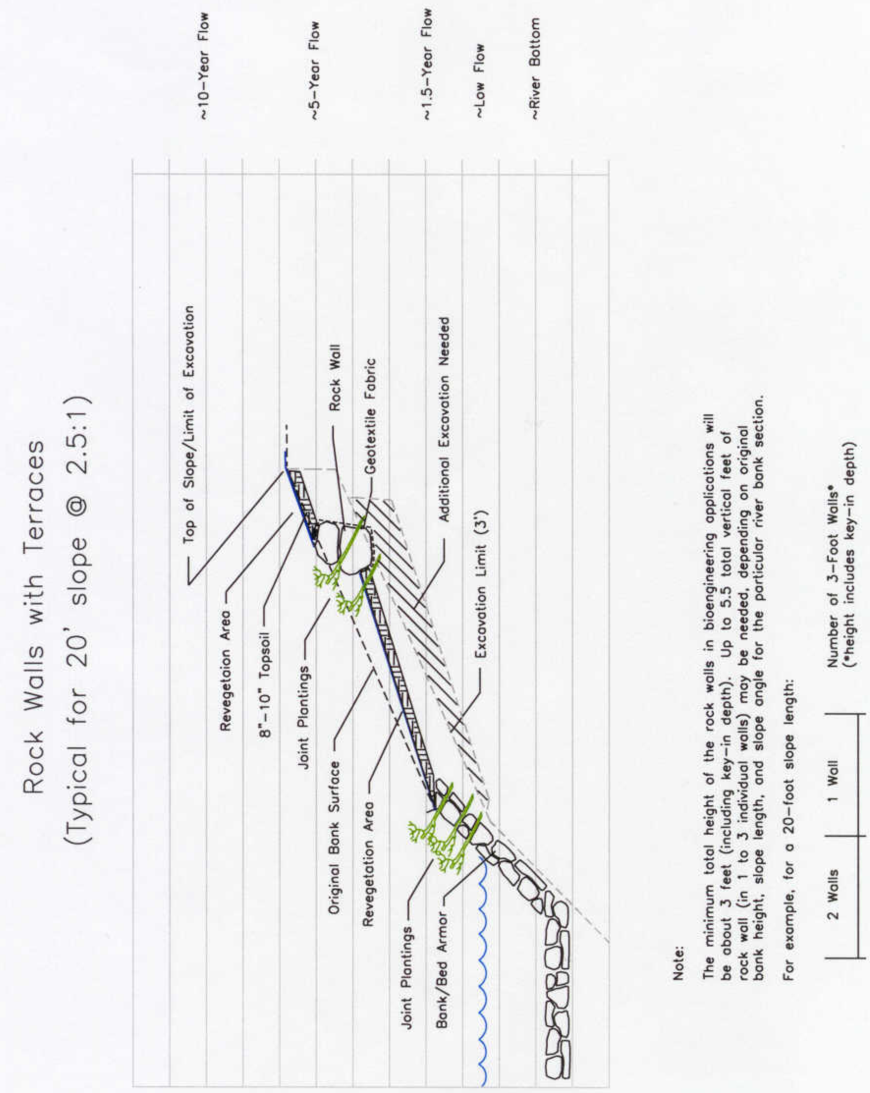
PREPARED BY: WOODLOT ALTERNATIVES, INC. 122 MAIN STREET, TOPSHAM, MAINE 04086 www.woodlotalt.com	SCALE: No Scale	Alternative 1 Vegetated Geogrid Reach 1 Bank Restoration Bioengineering Bank Stabilization
	DATE: 01/30/01	
	PROJ. NO. 100061.12	
	DWG. NAME: Bio_Det.dwg	REV.

Vegetated Geogrid

Considerations

- Can be installed on slopes up to 1H:1V.
- The soil lifts have a relatively high initial tolerance (2-4 years) of scour from flow velocity before the installed plants stabilize the slope.
- Requires stable foundation and bank toe support, and may require additional fill depending on existing grades.
- Plants in soil lifts provide or enhance slope drainage and help establish riparian vegetation community.
- Allows use of dormant cuttings between November and April.
- Containerized plants would be required during the growing season, April – Nov.
- Geogrids could be constructed any time of the year.
- Total costs for cuttings and containerized plants are approximately equal when storage, handling, shipping, and installation are considered.
- Refrigeration could also be used to extend construction period for cuttings but survivorship may be low
- Labor intensive to install, needs a skilled crew.
- Fabric can be natural (coir) or synthetic geotextile. Synthetics last longer but not as natural in appearance.
- Long-term slope stability depends on cutting and containerized plant establishment.
- Can be merged with other structures.
- Can be replanted but would require live stakes and/or containerized plants, which may cause some local geogrid instability.
- If cuttings are used plant diversity decreases. Only limited species are applicable for the given site conditions.
- "Rooted socks" could also be used but may add construction complexity (storage, planting dates unknown) and have limited success in other areas of western Massachusetts.



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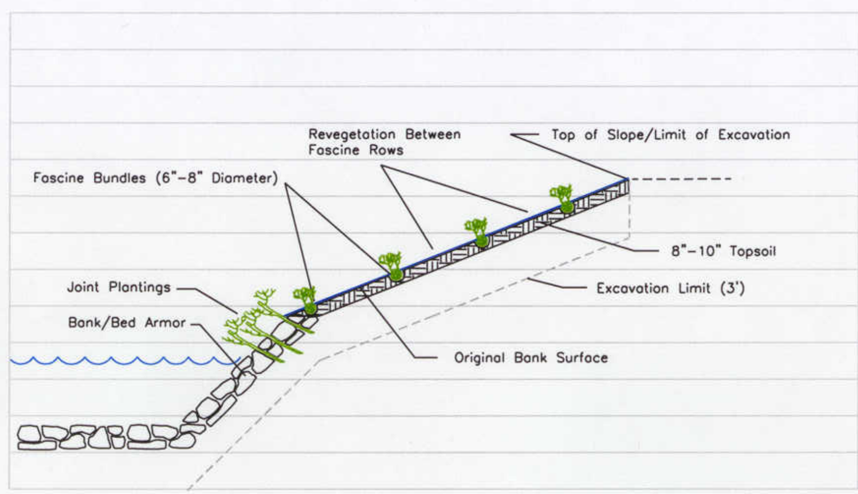
PREPARED BY: WOODLOT ALTERNATIVES, INC. 122 MAIN STREET, TOPSHAM, MAINE 04086 www.woodlotalt.com	SCALE: No Scale	Alternative 2
	DATE: 01/30/01	Rock Walls with Terraces
	PROJ. NO. 100061.12	Reach 1 Bank Restoration
	DWG. NAME: Bio_Det.dwg	Bioengineering Slope Stabilization
		REV.

Rock Walls with Terraces

Considerations

- Boulders (1-3 ft diameter) would be installed on the slope to form low (2 – 3 feet high) retaining walls and shallow ($\leq$ 3H:1V) terraces.
- One or two low walls would be needed, depending on the slope length and angle.
- Geotextile and soil compaction measures needed.
- May be a stability problem if rocks are not properly keyed in or anchored to the slope (key in depth approximately 1.5 to 2 feet).
- Requires a source of angular rock/boulders 1-3 feet in diameter. May increase shipping costs.
- Dormant cuttings and/or containerized plants can be used between boulders within the wall.
- Could be constructed anytime of the year.
- Requires skilled crew to build wall.
- Installation for cuttings should occur between November to April.
- Containerized plants would be required during the growing season (April to Nov) .
- To maintain 3-foot fill over the excavation limit, some additional excavation is required, which increases remediation costs.
- May be more difficult to merge with other bank stabilization measures (e.g., a slope with terraces to a uniform grade slope).
- Can be easily replanted if needed.
- May be used in areas where terraces currently exist and mimic natural floodplain geomorphology.
- If installed properly should provide long-term site stability with low maintenance.

Live Fascines
(Typical for 20' slope @ 2.5:1)



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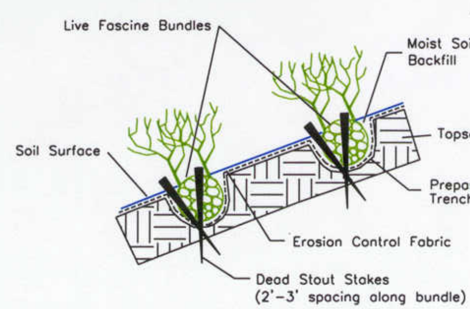
Note:

Live fascines have been shown to be effective on fill slopes up to 2.5:1 H:V. Fascines can be installed along the contour for slope stabilization and sediment trapping, or can be installed at an angle on the slope to also facilitate surface drainage. Slope sections between fascine rows would be revegetated with containerized plants and seeding. The spacing of fascines on the slope depends on the steepness as indicated below.

6-8 Feet	8-9 Feet	Spacing between fascine trenches installed on the contour
2.5:1 — 3:1	3:1 — 4:1	Slope Angle

INSTALLATION DETAIL



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SCALE: No Scale
DATE: 01/30/01
PROJ. NO. 100061.12
DWG. NAME: Bio_Det.dwg

Alternative 3
Live Fascines
Reach 1 Bank Restoration
Bioengineering Bank Stabilization

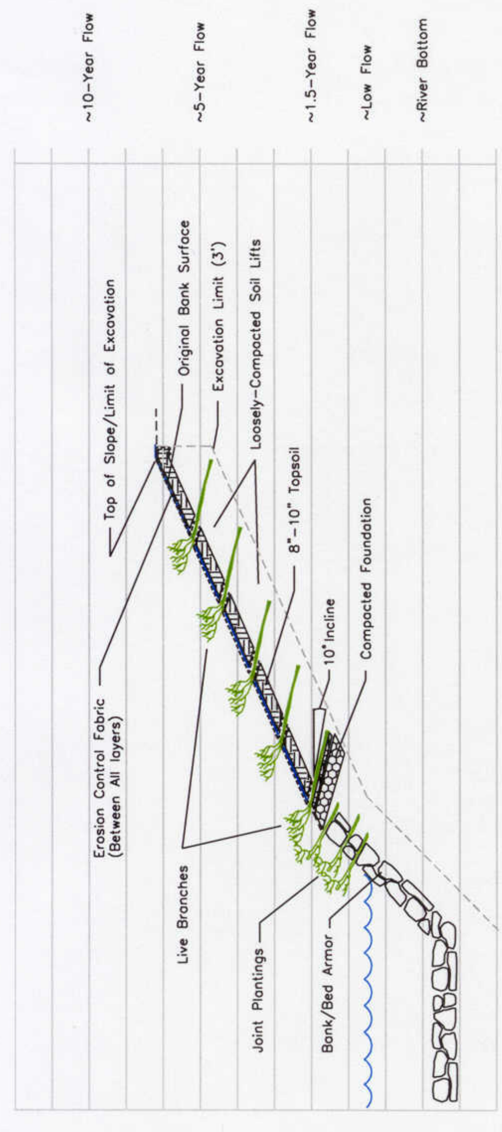
REV.

Live Fascines

Considerations

- Installed along the slope parallel to river, can provide dam-like trapping of sediments from bank surface erosion.
- Does not immediately protect slopes from flow velocities.
- Often installed in combination with erosion blankets to protect soil in between fascines.
- For site conditions (fill slopes with 3 ft of fill) they are appropriate on slopes less than 2.5H:1V.
- Requires a minimum amount of soil disturbance to install.
- Needs to be installed during dormant seasons unless refrigerated storage is available.
- Requires large amount of live cuttings. Storage needed.
- Installation of dormant cuttings occurs between November and April.
- Can be installed after banks have been reconstructed but trenches need to be dug prior to erosion blanket installation.
- Cuttings limited to narrow range of species (2 – 3) for the given site conditions and review of stabilization experiences in western Massachusetts.
- Containerized plants and seeding installed between rows of fascines to increase species diversity.
- Trained labor needed to install.
- Number of rows of fascines inversely proportional to steepness of slope.
- Long-term slope stability depends on fascine and containerized plant establishment.
- Can be easily replanted if needed.
- Relatively inexpensive.

Brush Layering
(Typical for 20' slope @ 2.5:1)



Note:
Brush Layering has been shown to be effective on slopes up to 1.5:1 H:V. Brush layers are typically installed along the contour for slope stabilization and sediment trapping, and can be installed on cut or fill slopes. The spacing of brush layers on the slope depends on the steepness as indicated below.

	2-3 Feet	3-4 Feet	4-5 Feet	5-6 Feet	Slope distance between brush layers installed on the contour
1.5:1	2:1	2.5:1	3:1	4:1	Slope Angle

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www.woodlotalt.com

SCALE: No Scale
DATE: 01/30/01
PROJ. NO. 100061.12
DWG. NAME: Bio_Det.dwg

Alternative 4
Brush Layering
Reach 1 Bank Restoration
Bioengineering Bank Stabilization

REV.

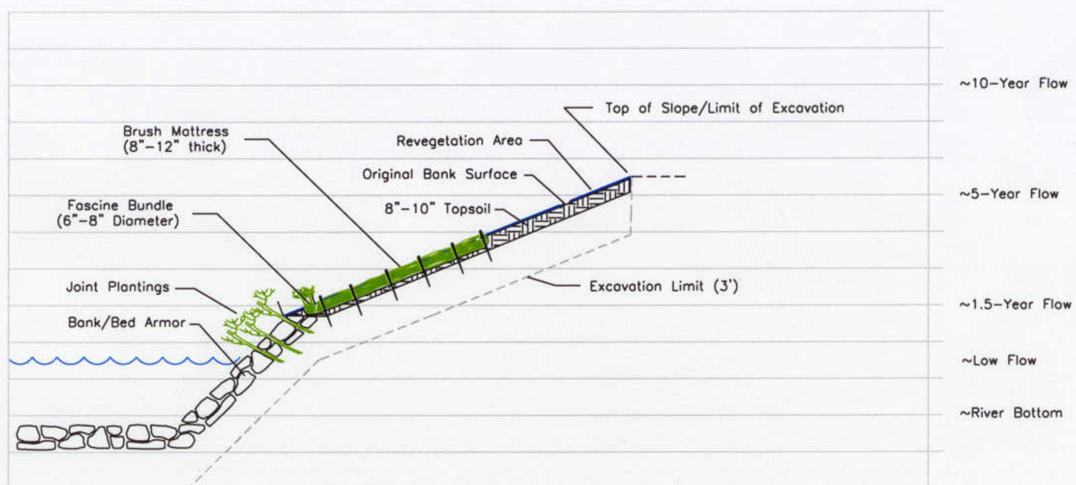
Brush Layering

<u>Considerations</u>

- Similar to vegetated geogrid but does not provide as much immediate protection from scouring at higher flows.
- Typically appropriate for slopes less than 2H:1V.
- Requires large amounts of live plant materials and labor to install.
- Limited to dormant seasons unless refrigerated storage is available.
- Provides some initial protection from higher flows and surface erosion (<3 years) by adding bank roughness/trapping of sediments.
- Depends on vegetation establishment from cuttings for long-term bank erosion.
- Installation of dormant cuttings occurs between November and April.
- Cuttings limited to narrow range of species (2 – 3) for the given site conditions and review of stabilization experiences in western Massachusetts
- Trained labor needed to install.
- Number of rows inversely proportional to steepness of slope.
- Long-term slope stability depends on brush layer and containerized plant establishment.
- Difficult to replant if needed.

Brush Mattress

(Typical for 20' slope @ 2.5:1)

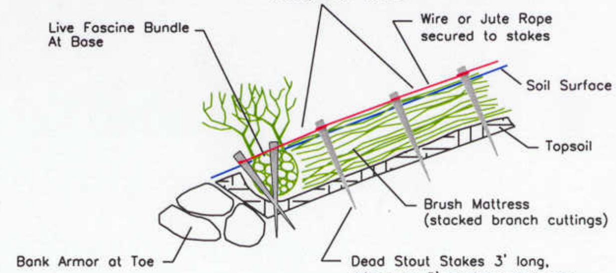


DRAFT CONFIDENTIAL

Note:

Brush Mattresses are effective on slopes up to 2:1, and form an immediate protective cover on river bank. Properly-installed mattresses will sprout numerous individual plants during the first growing season. Branch layers must be installed in contact with soil, and the basal (or butt) ends should be covered to prevent drying out. A fascine bundle can be used at the base to help keep mattress in place until rooting occures. Branches within the mattresses should be 4'-10' long. Multiple mattresses can be installed on long slopes, as long as lower mattresses overlap upper ones by at least 12 inches. Mattresses must be tied to slope using wire or jute rope secured to stakes in a criss-cross fashion. Mattresses should be covered loosely with topsoil, leaving buds from the top layer of branches exposed. Brush mattresses must be installed only in the dormant season.

INSTALLATION DETAIL



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SCALE: No Scale

DATE: 01/19/01

PROJ. NO. 100061.12

DWG. NAME: Bio_Det.dwg

Alternative 5
Brush Mattress
Reach 1 Bank Restoration
Bioengineering Bank Stabilization

REV.

Brush Mattress

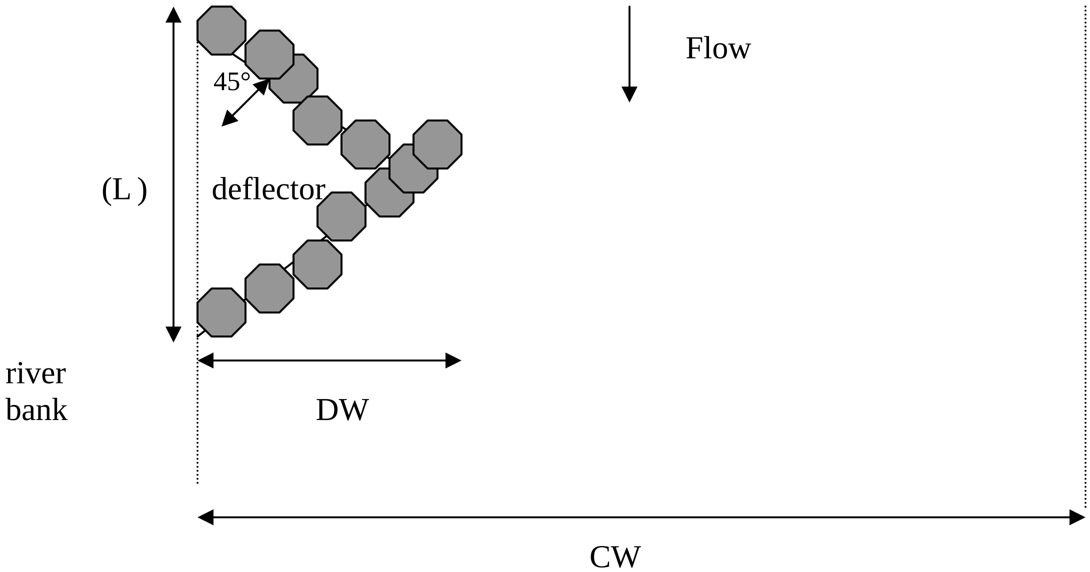
Considerations

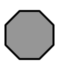
- Forms an immediate, protective cover over the streambank.
- Typically appropriate for slopes less than 2H:1H.
- Requires large amounts of live plant materials and labor to install.
- Limited to dormant seasons unless refrigerated storage is available.
- Depends on vegetation establishment from cuttings for long-term bank erosion.
- Installation of dormant cuttings and/or rooted plants occurs between November and April.
- Cuttings limited to narrow range of species (2 – 3) for the given site conditions and review of stabilization experiences in western Massachusetts
- Relatively simple design.
- Containerized plants and seeding installed above the brush mattress to increase species diversity.
- Trained labor needed to install.
- Length of mattress proportional to steepness of slope.
- Long-term slope stability depends on brush layer and containerized plant establishment.
- Requires equipment to replant.

APPENDIX I

DESIGN CALCULATIONS FOR WING DEFLECTORS

Single Wing Deflector Calculations
[Woodlot Alternatives Inc., 12/29/01]



 = Boulders (2 to 2.5 ft dia) For rock sizes within the deflector see Sheet 20.

(1) Effective deflector width (DW) = 0.3 * Low-flow channel width (CW)
[0.3 was used to limit backwater effects while meeting restoration objectives (Fischenich (2001a)].

(2) Deflector width and length:

@ STA 504 to 507 Average CW = 62.5 ft → DW ~ 20 ft
Deflector length (L) = (20 ft – 3 ft) * 2= ~ 35 ft

@ STA 510 to 514 Average CW = 59.0 ft → DW ~ 18 ft
Deflector length (L) = (18 ft – 3 ft) * 2 =~ 30 ft

[Low-flow channel widths determined from field measurements (Woodlot Alternatives (2000)]

APPENDIX J

BOULDER STABILITY CALCULATIONS

BOULDER STABILITY ASSESSMENT
[Woodlot Alternatives, Inc (2/15/02)]

A. Critical Shear Velocity Approach (Fischenich and Allen, 2000)

(1) HEC-RAS Results (Hart Crowser 3/01)

Assume Elm Street Bridge in not re-constructed

Avg shear stress (τ_{avg}) and velocity @ design flood (Q_{10} = 4375 cfs)

Occurs @ STA 502:

Avg Vel= 6.5 ft/s

τ_{avg} = 0.4 lbs/ft^2

(2) Calculate critical shear velocity ($V_{*}c$)

Assume $\tau_{max} = 3 * \tau_{avg}$; Using τ_{avg} = 0.4 lbs/ft^2 ; τ_{max} = 1.2 lbs/ft^2

$V_{*}c = (gRs)^{1/2}$ where g is the acceleration of gravity 32.2 ft/sec^2 ;
R is the hydraulic radius;
S is slope (friction)

$\tau = \gamma Rs$ where γ is the specific weight of water 62.3 lbs/ft^3

solving for R; $R= \tau/\gamma s$ and substituting into critical velocity equation:

$V_{*}c = (g\tau/\gamma)^{1/2} = [(32.3 \text{ ft/sec}^2 * 1.2 \text{ lbs/ft}^2)/ 62.4 \text{ lbs/ft}^3]^{1/2}$

$V_{*}c$ = 0.8 ft/s

Using Table 2.9 of Fischenich and Allen (2000)

Diameter of stable rock (d_{stable})= 3.0 in (approx) =>small cobble

(3) Assume Factor of Safety of 2.0

$V_{*}c$ = 0.8 ft/sec * 2 = 1.6 ft/sec [Use Table 2.9]

d_{stable} ~12 in (small boulder)

Therefore, boulder sizes proposed for the final design are 2.5 ft diameter (min), which is larger than the minimum size estimated above, and would be stable at the design flood.

Note: Another method to determine boulder stability would be to use Table 7.7 (Fischenich and Allen, 2000), which estimates the threshold critical velocity and critical shear stresses that would be needed to move various sediment sizes. This method essentially yields similar results as the above analysis because both Table 2.9 and 7.7 are based on the same principles (i.e., the forces acting on the boulder). For example, boulders used in the final design have average diameters of 2.5 ft or greater and to move this size would require a critical shear stress of approximately 10 lbs/ft^2 (Table 7.7) or a critical shear velocity of approximately 2.4 ft/sec (Table 2.9). The critical shear velocity calculated above is less than this value and, therefore, this boulder size would be stable.

APPENDIX K

WING DEFLECTOR SCOUR ANALYSES

Wing Deflector Scour Analyses

[Hart Crowser, Inc; 7/9/01]

Table K-1 Analysis of Rock Deflector as Bend

	Input	Storm Year		
		0.5-yr		
Rock Deflectors				
	Station	503+50	TK =1	TK=1.5
	Velocity	5.23	5	5.23
	Flow Depth	5.9	6	5.9
	D30		0.36	0.32
	D100		9"	9"

Assessment of Riprap Size	Rock Deflectors
Side Slope	5
Bend Radius	100
Water Surface Width	60
Layer Thickness (xD100)	1
Unit Weight of Stone	165
Safety Factor	1.1
Station Start	500+00
Station End	514+00

Notes:
1. For Rock Deflectors, worst case was assumed to be the station which had the highest velocity and lowest depth for storms between 0.5- and 2-yr return period.
2. Bend radius and water surface width estimated between rock deflectors
3. Side slope of 5:1 at rock deflector assumed because scour potential would be on bed in vicinity of deflector.
4. D30 calculated from Riprap 15 program. D100 chosen from Table 3-1, EM-1110-2-1601 (USACE, 1994), assuming that the D30 of the riprap gradation > D30 calculated.

Table K-2 Contraction Scour Through Rock Deflectors

Parameter		Storm Interval		
		0.125-yr	0.25-yr	.5-yr
Upstream flow stage[1]		972.9	974.6	975.7
Channel Invert		969.4	969.4	969.4
Upstream flow depth[2]	y1	3.5	5.2	6.3
Upstream flow width[3]	W1	87	93	96
Flow width through constriction[4]	W2	62	68	71
Representative Grain Size	D30	111	111	111
Shear stress[5]		0.09	0.12	0.14
Critical shear stress[6]		1	1	1
	Bc/B	0.7	0.7	0.7
	Tc/T	11.1	8.3	7.1
	Δz/h	-0.52	-0.47	-0.44
Scour Depth[7]	**Δz**	**-1.8**	**-2.5**	**-2.8**

Notes
Parameters were chosen from HEC-RAS analysis of current conditions to represent worst-case conditions in reach where rock deflectors were to be placed (stations 503+00 to 507+00 & 510+00 to 513+50).
1. Maximum stage (ft) estimated from station 503+00
2. Flow depth (ft) = Upstream stage - Channel Invert at station 503+00
3. Width (ft) = Top Width at station 503+00
4. Constricted width (ft) assumes deflectors extend 25 feet into flow. Constricted Width =Width - 25 feet
5. Maximum shear stress (lb/ft^2) estimated from station 503+50
6. Critical shear stress (lb/ft^2) estimated from Table X-3 Incipient Motion Conditions
7. Scour Depth (ft) calculated based on the Gill Equation (Gill, 1972)

Gill Equation

Δz/h=((Bc/B)^(-6/7)*((Bc/B)^(-2/3)*(1-τc/τ)+τc/τ)^(-3/7)-1)

Δz = contraction scour depth
h = approach water depth
Bc = constricted channel width
B = approach channel width
τc = critical shear stress
τ = shear stress (obtained from HEC-RAS analysis)

Predicted scour <0 suggest that scour is unlikely.
Flows greater than the 0.5-year were not evaluated. It was assumed that this equation did not correctly represent conditions at higher flows because it assumes all flow passes through the constriction. It was assumed that at higher flows the deflectors acted more as roughness elements than constrictions.

Table K-3 Incipient Motion Conditions

Critical Shear Stress Calculation

Unit Grain Size		φ	θ	τcritical Bottom				τcritical Side Slopes				AVERAGE CRITICAL SHEAR STRESS	
				S and H Tc	Lane(Fig. 7.7) Tc fines	clear	Shields Tc		S and H Tc	Lane Tc	Shields Tc	BOTTOM	SIDES
mm	ft	fig 7.16		lb/ft2	lb/ft2	lb/ft2	lb/ft2	K	lb/ft2	lb/ft2	lb/ft2		
0.1	3.3E-04	0.49	0.46	0.003	0.08	0.02	0.002	0.104	0.0003	0.0052	0.000	0.026	0.002
1	3.3E-03	0.51	0.46	0.012	0.09	0.03	0.016	0.167	0.0021	0.0100	0.003	0.037	0.005
10	3.3E-02	0.56	0.46	0.203	0.25	0.15	0.159	0.322	0.0652	0.0644	0.051	0.190	0.060
25	8.2E-02	0.57	0.46	0.507	0.5	0.4	0.397	0.350	0.2042	0.1576	0.139	0.451	0.167
50	1.6E-01	0.59	0.46	1.014	1	0.8	0.794	0.403	0.4084	0.3626	0.320	0.902	0.364
100	3.3E-01	0.66	0.46	2.027	2.5	1.5	1.587	0.528	1.0705	1.0562	0.838	1.904	0.988

Physical Parameters

g ft/s2	γ lb/ft3	γs lb/ft3	for Shields Eqn βs
32.2	62.4	165.36	0.047

Notes:
Grain size for which incipient motion was calculated
φ = internal friction angle of material
θ = side slope angle (radians). Assumed 2:1 slope q = arctan(1/2)
Equations from Simons and Senturk, 1992.
Shulits and Hill (S and H)
Tc = 0.0215*Ds^0.25 if 0.0003 < Ds <0.0009,
Tc = 0.315*Ds^0.633 if 0.0009 < Ds <0.0018,
Tc = 16.8*Ds^1.262 if 0.0018 < Ds <0.022,
Tc = 6.18*Ds if Ds >0.022
Where Ds = characteristic grain size
2. Lane, Critical Tractive Force
Shear stress read from Figure 7.7 in Simons and Senturk, 1992.
3. Sheilds Equation
Tc=γs'Ds*βs

Equations calculate critical shear stress for bed. Critical shear stress on banks estimated as follows:
Tc sides = Tcbottom*K
where K = COS(θ)*(1-(TAN(θ))^2/(TAN(φ))^2)

Table K-4 Rock Deflector Spill Over

Mat'l Size	Head Drop[1] ft	Velocity[2] ft/s	Velocity Head ft	H[3] ft	q[4] cfs/ft	q[4] cms/m	H[5] m	hd[6] m	Schoklitsch ds[7] m	Schoklitsch S[8] ft	Jager ds[9] m	Jager S[8] ft	Avg S ft
A	2.00	6.0	0.6	2.6	5.7	0.53	0.8	1.07	0.65	-1.36	0.78	-0.95	-1.15
B	2.00	6.0	0.6	2.6	5.7	0.53	0.8	1.07	0.60	-1.54	0.71	-1.18	-1.36
C	2.00	6.0	0.6	2.6	5.7	0.53	0.8	1.07	0.55	-1.68	0.66	-1.35	-1.51

Flow Parameters

Q cfs	B ft	g
500.0	87	32.2

Material Properties

Mat'l Size	D100 inch	D90 ft	D90 mm
A	9	0.53	159
B	12	0.70	210
C	15	0.88	264

Notes
1. Approximate head drop through sheet pile constriction.
2. Velocity based on approximate maximum velocity in design reach for 0.5-, 0.75-, 1-, 1.5-, and 2-yr storms. It is assumed that larger storms will drown out the deflectors
3. Total Head difference = head drop + velocity head
4. q = flow/unit width
5. H in meters
6. hd = downstream water depth
7. ds =scoured water surface depth downstream = 4.75*H^0.2*q^0.5/D90^0.32
8. S = scour depth = hd - ds
9. ds =scoured water surface depth downstream ==6*H15^0.25*G15^0.5*(I15/K6)^0.3333
10. Equations listed in Simons and Senturk, 1992

APPENDIX L

SHEET PILE DESIGN CALCULATIONS

SCENARIO I

SHEET PILE DESIGN
SHEETING LINE ALONG CENTERLINE OF RIVER (3-FT CUT)

CLIENT/SUBJECT GE - Pittsfield W.O. NO. ______

TASK DESCRIPTION Sheet Pile Design TASK NO. ______

PREPARED BY WLD DEPT 1274 DATE 3/1/02

MATH CHECK BY SW DEPT 1382 DATE 3/8/02

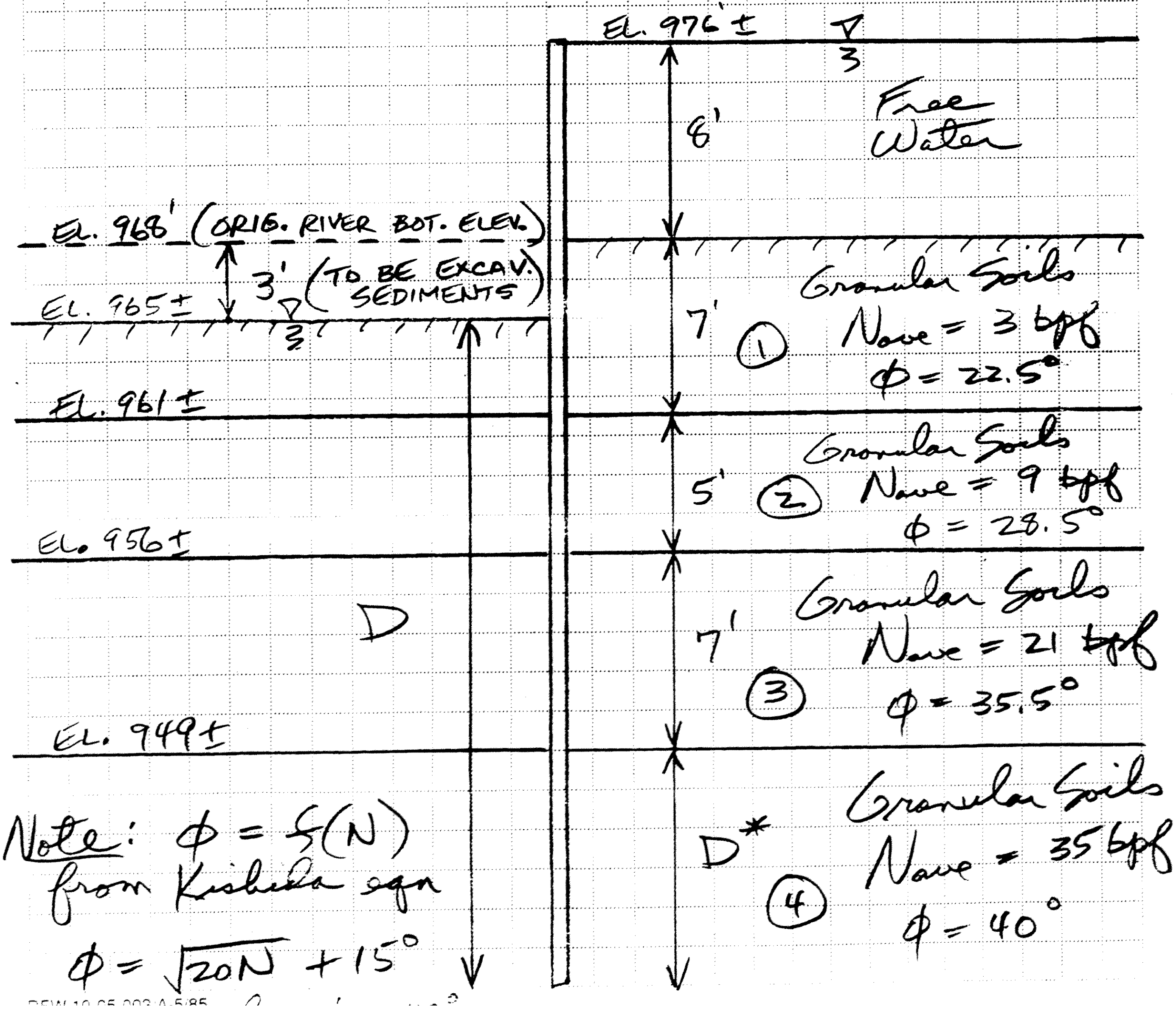
METHOD REV. BY ______ DEPT ______ DATE ______

APPROVED BY ______ DEPT ______ DATE ______

SCENARIO I
SHEET PILE DESIGN

SHEETING LINE ALONG ℄ RIVER (3' CUT)

Based on relevant borings within river channel and along river bank, the selected design cross section is:



Note: $\phi = f(N)$ from Kishida eqn

$\phi = \sqrt{20N} + 15^\circ$

WESTON
MANAGERS DESIGNERS/CONSULTANTS

SHEET 2 of ____

CLIENT/SUBJECT ____________________ W.O. NO. ________

TASK DESCRIPTION ____________________ TASK NO. ________

PREPARED BY __________ DEPT ______ DATE ______

MATH CHECK BY __________ DEPT ______ DATE ______

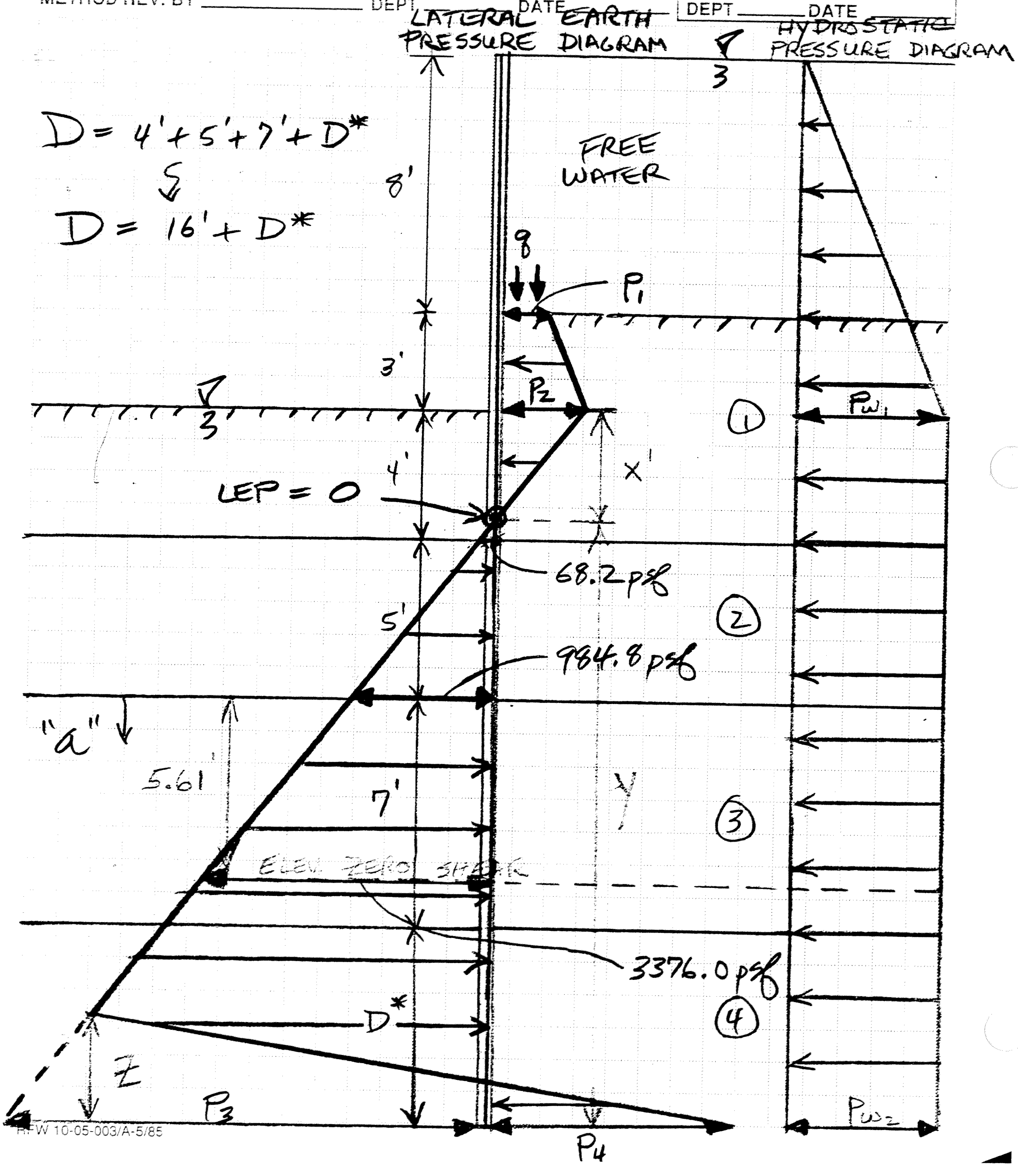
METHOD REV. BY __________ DEPT ______ DATE ______

APPROVED BY

DEPT ______ DATE ______

$D = 4' + 5' + 7' + D^*$

$D = 16' + D^*$



RFW 10-05-003/A-5/85

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SHEET 3 of ____

CLIENT/SUBJECT ____ W.O. NO. ____
TASK DESCRIPTION ____ TASK NO. ____
PREPARED BY ____ DEPT ____ DATE ____
MATH CHECK BY ____ DEPT ____ DATE ____
METHOD REV. BY ____ DEPT ____ DATE ____

APPROVED BY ____
DEPT ____ DATE ____

$P_{w_1} = 11' \times 62.4\text{ pcf} = 686.4\text{ psf}$

$P_{w_2} = P_{w_1} = 686.4\text{ psf}$

K_A & K_p VALUES: (See Figure on pg 3A from USS Sheet Pile Design Manual)

1.) Stratum 1

$K_A: \quad \phi = 22.5^\circ$

$\beta/\phi = 0^\circ$

$\delta/\phi = .5$ (ASSUMED)

$$\boxed{K_A = .42}$$

$K_p: \quad \phi = 22.5^\circ$

$\beta/\phi = 0^\circ \rightarrow K_{p_u} = 3.6$

$\delta/\phi = .5 \rightsquigarrow R = .835$

$K_p = 3.6(.835) = 3.01$

$$\boxed{K_p = 3.01}$$

3A

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$								
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

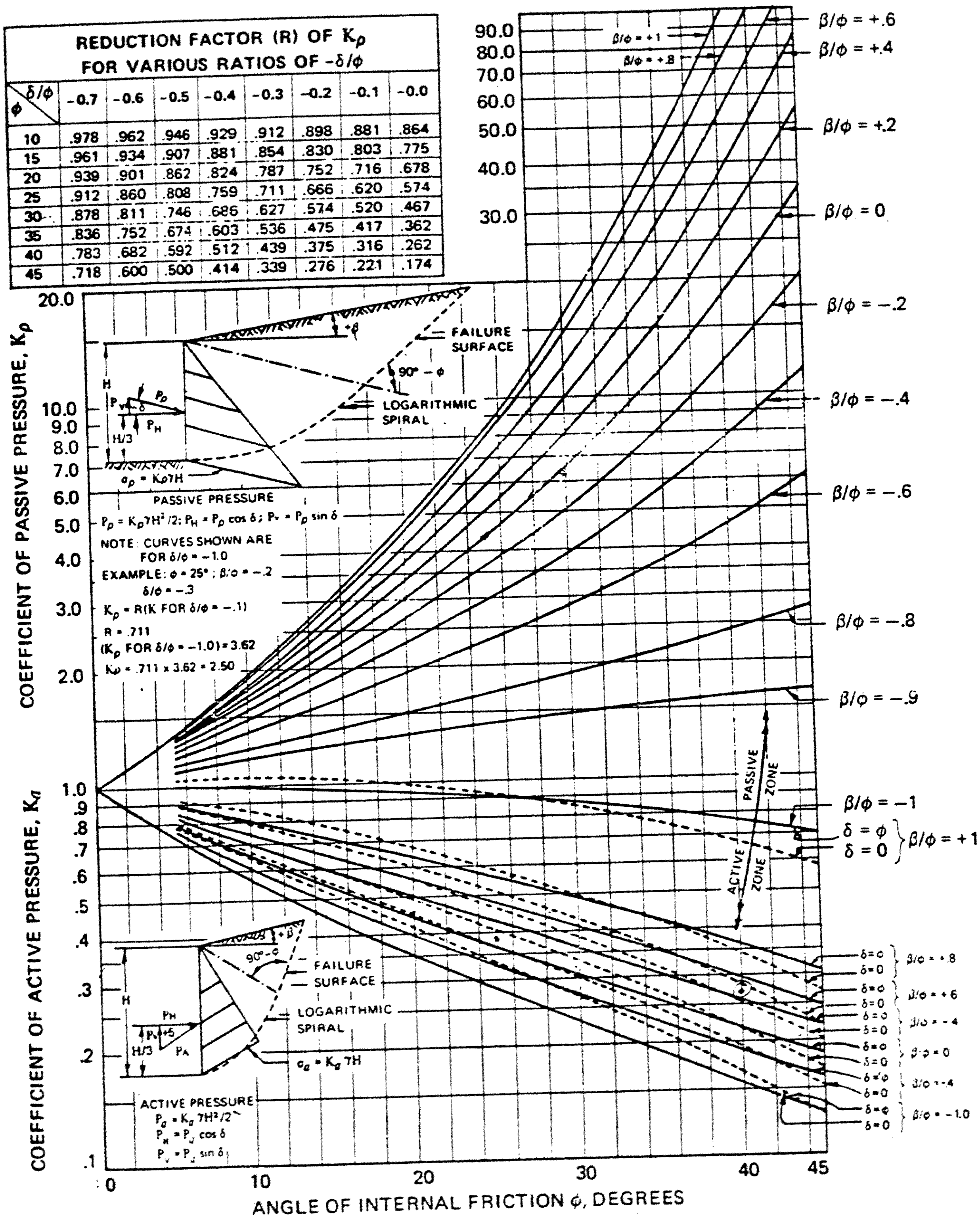

Fig. 5(a) – Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[21])

CLIENT/SUBJECT ___ W.O. NO. ___

TASK DESCRIPTION ___ TASK NO. ___

PREPARED BY ___ DEPT ___ DATE ___ APPROVED BY

MATH CHECK BY ___ DEPT ___ DATE ___

METHOD REV. BY ___ DEPT ___ DATE ___ DEPT ___ DATE ___

2.) Stratum 2

K_A : $\phi = 28.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$\boxed{K_A = .34}$

K_p : $\phi = 28.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$K_{pu} = 5.7$ $R = .765$

$\boxed{K_p = 5.7(.765) = 4.36}$

3.) Stratum 3

K_A : $\phi = 35.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$\boxed{K_A = .26}$

K_p : $\phi = 35.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$K_{pu} = 11.5$ $R = .666$

$\boxed{K_p = 11.5(.666) = 7.66}$

WESTON
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SHEET 5 of

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4.) Stratum 4

K_A: $\phi = 40°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$\boxed{K_A = .22}$

K_P: $\phi = 40°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$K_{P_u} = 18.0$ $R = .592$

$K_P = 18.0\,(.592) = 10.65$

$\therefore \boxed{K_P = 10.65}$

Unit Weights (γ_{SAT}):

$\gamma_{SAT} = f(N)$

– see Appendix A; From this data:

Soil Stratum	γ_{SAT} (pcf)
1	93
2	108
3	120
4	127.5

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Then $P_1 = q K_A = [8' \times 62.4\,pcf](.42) = 210\,psf$

surcharge load due to wt of 8' of free water

See pg 2 Figure

$P_2 = P_1 + (93 - 62.4\,pcf)(3')(.42) = 249\,psf$

Calculate:

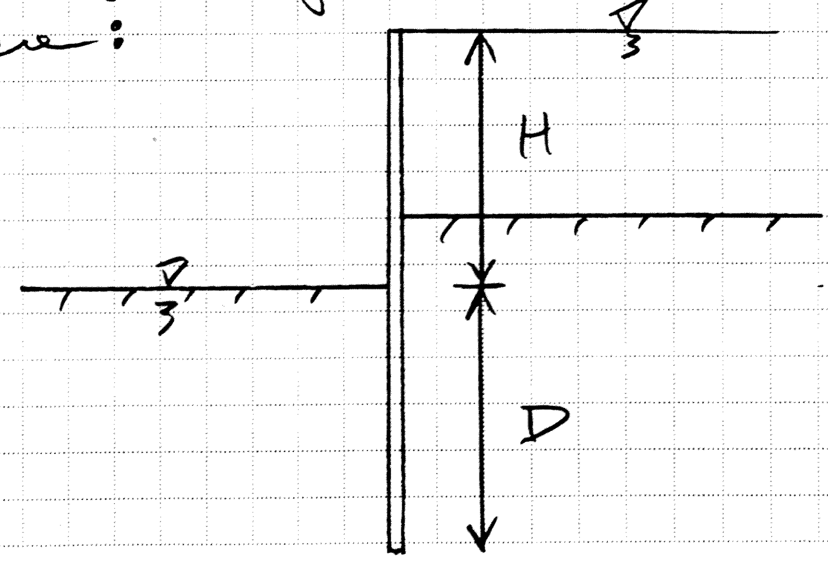
see pg 2

$P_3 = P(D) - A(H+D)$ → See App B, pg B-3

Passive LEP calc based on all strata within Distance D to bottom of sheeting

Active LEP calc based on all strata within Distance (H + D) to bottom of sheeting

where:



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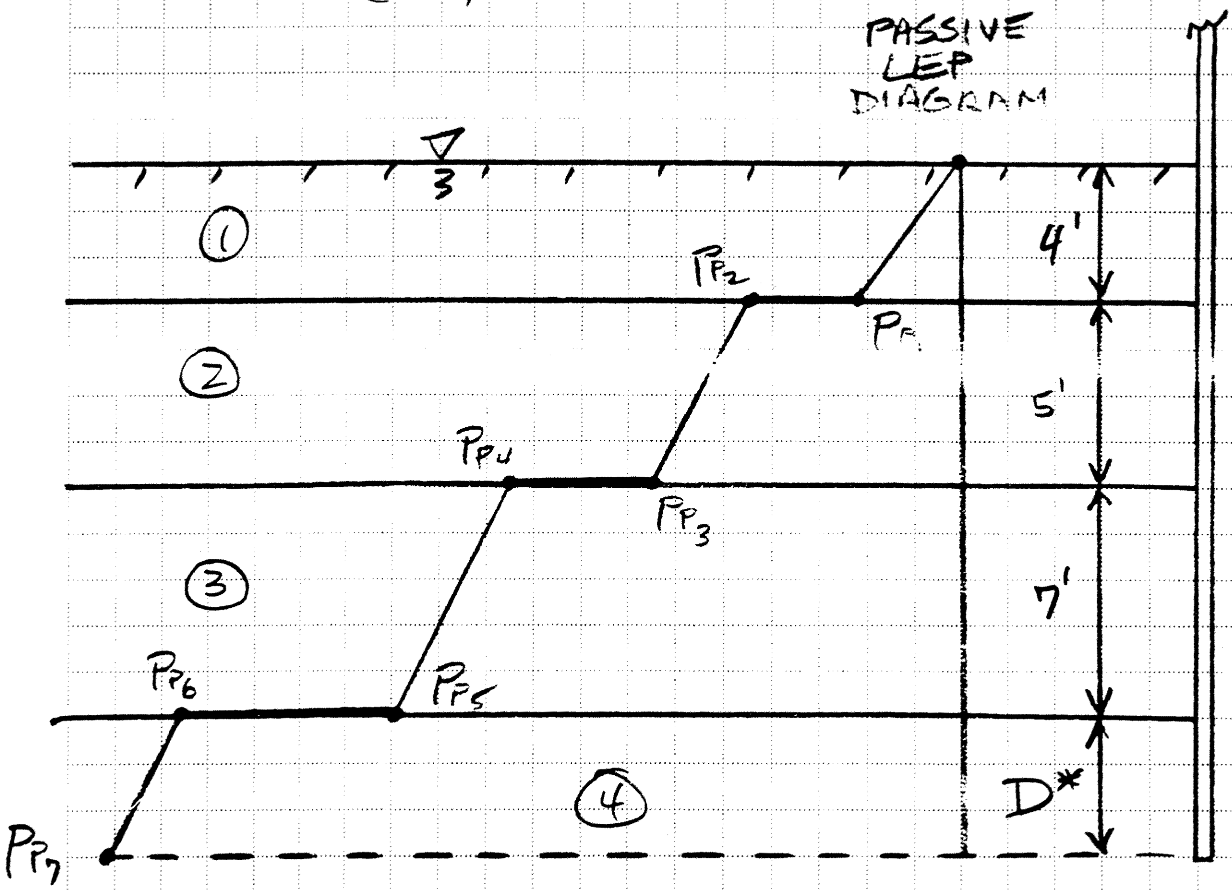
Then P(D) is calculated as:



$$P_{P_1} = \underset{pcf}{(93-62.4)}(4')(3.01) = 368.4 \text{ psf}$$

$$P_{P_2} = \left[(93-62.4)(4')\right](4.36) = 533.7 \text{ psf}$$

$$P_{P_3} = 533.7 + \underset{pcf}{(108-62.4)}(5')(4.36) = 1527.7 \text{ psf}$$

$$P_{P_4} = \left[\underset{pcf}{(93-62.4)}(4') + \underset{pcf}{(108-62.4)}(5')\right](7.66) = 2684.1 \text{ psf}$$

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$$P_{P_5} = 2684.1 + (120 - 62.4)(7')(7.66)$$

$$= 5772.6 \text{ psf}$$

$$P_{P_6} = \Big[(\underset{\text{pcf}}{93} - 62.4)(4') + (\underset{\text{pcf}}{108} - 62.4)(5') + (\underset{\text{pcf}}{120} - 62.4)(7')\Big](10.65)$$

$$= 8025.8 \text{ psf}$$

$$P_{P_7} = 8025.8 + (127.5 - 62.4) D^* (10.65)$$

$$= 8025.8 + 693.3 D^*$$

Also, A(H+D) is calculated as:

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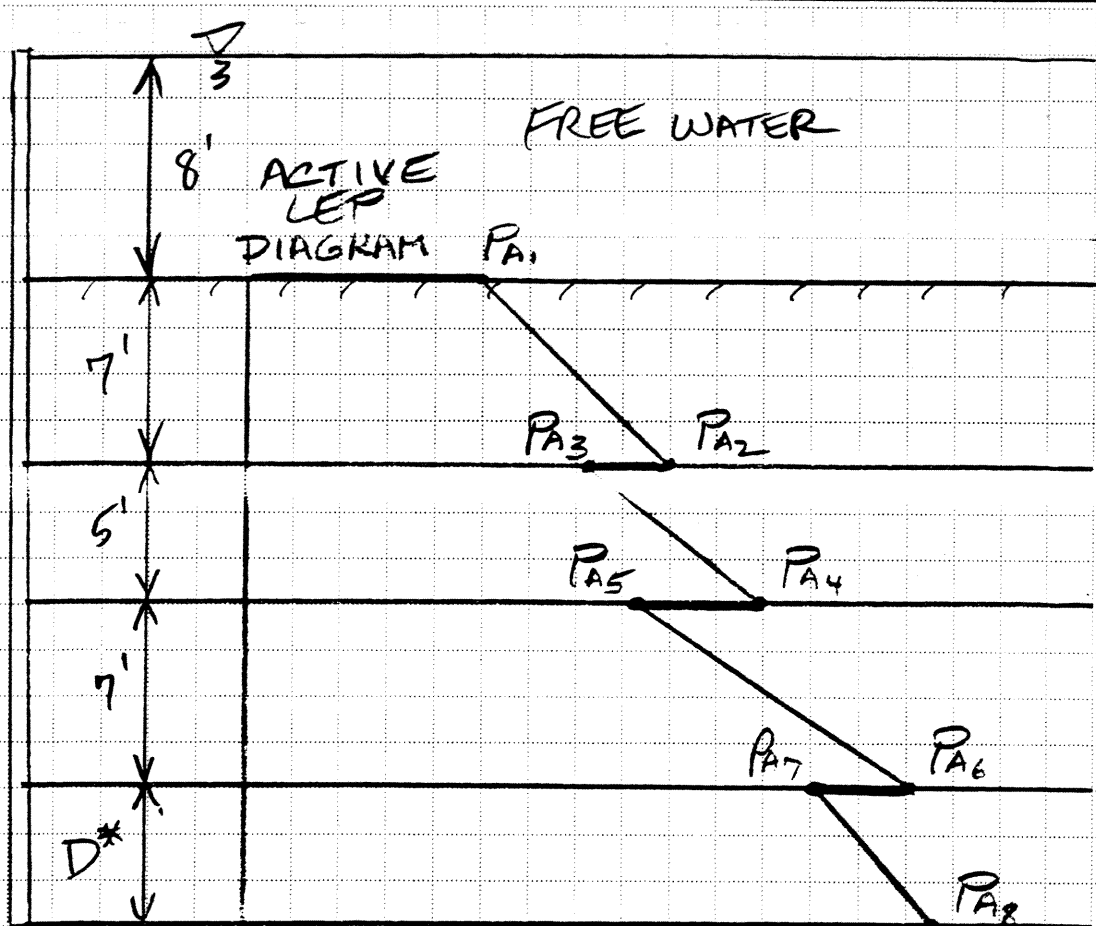

$$P_{A_1} = q K_A = (8' \times 62.4 \text{ pcf})(.42) = 210 \text{ psf}$$

$$P_{A_2} = 210 + (93 - 62.4 \text{ pcf})(7')(.42) = 300 \text{ psf}$$

$$P_{A_3} = [8 \times 62.4 + (93 - 62.4)(7')](.34) = 242.6 \text{ psf}$$

$$P_{A_4} = 242.6 + (108 - 62.4)(5')(.34) = 320.1 \text{ psf}$$

$$P_{A_5} = [8 \times 62.4 + (93 - 62.4)(7) + (108 - 62.4)(5)](.26)$$
$$= 244.8 \text{ psf}$$

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$$P_{A_6} = 244.8 + (120 - 62.4)(7')(.26)$$
$$= 349.6 \text{ psf}$$

$$P_{A_7} = \left[8 \times 62.4 + (93 - 62.4)(7) + (108 - 62.4)(5) + (120 - 62.4)(7)\right](.22) = 295.8 \text{ psf}$$

$$P_{A_8} = 295.8 + (127.5 - 62.4)(D^*)(.22)$$
$$= 295.8 + 14.3D^*$$

$$\therefore P_3 = P(D) - A(H+D)$$

see pg 2

$$= P_{P_7} - P_{A_8} = [8025.8 + 693.3D^*] - [295.8 + 14.3D^*]$$

see pgs 7 + 8   see pgs 9 + above

$$\therefore \boxed{P_3 = 7730 + 679D^*}$$

Calculate:

see pg 2 $\rightarrow$ $P_4 = P(H+D) - A(D)$ $\rightarrow$ see App B, pg B-3

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$P(H+D)$ is calculated as follows with reference to Fig on pg 9 where P_A values are replaced by P_P values calc. using K_p values:

$$P_{P_1} = qK_p = (8 \times 62.4)(3.01) = 1502.6 \text{ psf}$$

$$P_{P_2} = 1502.6 + (93-62.4)(7)(3.01) = 2147.3 \text{ psf}$$

$$P_{P_3} = \left[8 \times 62.4 + (93-62.4)(7')\right](4.36) = 3110.4 \text{ psf}$$

$$P_{P_4} = 3110.4 + (108-62.4)(5')(4.36) = 4104.5 \text{ psf}$$

$$P_{P_5} = \left[8 \times 62.4 + (93-62.4)(7) + (108-62.4)(5)\right](7.66)$$
$$= 7211.1 \text{ psf}$$

$$P_{P_6} = 7211.1 + (120-62.4)(7)(7.66)$$
$$= 10299.6 \text{ psf}$$

$$P_{P_7} = \left[8 \times 62.4 + (93-62.4)(7) + (108-62.4)(5) + (120-62.4)(7)\right](10.65)$$
$$= 14320.0 \text{ psf}$$

$$P_{P_8} = 14320 + (127.5-62.4)(D^*)(10.65)$$

$$= 14320 + 693.3D^*$$

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A (D) is calculated as follows with reference to Fig on pg 7 where P_P values are replaced by P_A values calc. using K_A values:

$$P_{A_1} = (93-62.4)(4')(.42) = 51.4 \text{ psf}$$

$$P_{A_2} = [(93-62.4)(4)](.34) = 41.6 \text{ psf}$$

$$P_{A_3} = 41.6 + (108-62.4)(5')(.34) = 119.1 \text{ psf}$$

$$P_{A_4} = [(93-62.4)(4') + (108-62.4)(5')](.26) = 91.1 \text{ psf}$$

$$P_{A_5} = 91.1 + (120-62.4)(7')(.26) = 195.9 \text{ psf}$$

$$P_{A_6} = [(93-62.4)(4) + (108-62.4)(5) + (120-62.4)(7)](.22) = 165.8 \text{ psf}$$

$$P_{A_7} = 165.8 + (127.5-62.4)(D^*)(.22) = 165.8 + 14.3D^*$$

$$\alpha \quad P_4 = P(H+D) - A(D) = P_{P_8} - P_{A_7}$$

see pg 2 see pg 11 see above

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$$\therefore \; P_4 = \left[14320 + 693.3D^*\right] - \left[165.8 + 14.3D^*\right]$$

$$\therefore \; \boxed{P_4 = 14154.2 + 679D^*}$$

Now determine distance x' (see pg 2) as follows:

assumes dist x' lies within stratum ①; will check later

$$\longrightarrow P_2 - \gamma'_{①}\left[K_{P①} - K_{A①}\right]x' = 0$$

$$\therefore \; x' = \frac{P_2}{\gamma'_{①}\left[K_{P①} - K_{A①}\right]}$$

$$= \frac{249 \text{ psf}}{(93 - 62.4)_{\text{pcf}}(3.01 - .42)}$$ ← see pg 6

$$x' = 3.14' < 4'$$ OK

i.e., x' lies within stratum ① & above eqn is valid

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Then $\sum F_H = 0$:

$$\frac{1}{2}\left(\underset{psf}{686.4}\right)(11') + \left(\underset{psf}{686.4}\right)(4'+5'+7')$$

$$+ \left(\underset{psf}{686.4}\right)(D^*) + 3'\left(\frac{210psf + 249psf}{2}\right)$$

$$+ \frac{1}{2}(249psf)(3.14') - \frac{1}{2}\left[7730 + 679D^*\right]y$$

see pg 2

$$+ \frac{1}{2}(z)\left[(7730 + 679D^*) + (14154.2 + 679D^*)\right]$$

$$= 0$$

see pg 2

$$3775.2 + 10982.4 + 686.4D^* + 688.5$$

$$+ 390.9 - 3865y - 339.5D^*y$$

$$+ 10942.1z + 679zD^* = 0$$

Eqn is $f(D^*, z + y)$; eliminate one variable noting that (see pg 2):

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$$16' + D^* = \underbrace{3.14'}_{x'} + y$$

$$y = D^* + 12.86$$

$\therefore$ Sub in above eqn & simplify:

$$15837 + 686.4D^* - 3865(D^* + 12.86)$$
$$- 339.5D^*(D^* + 12.86) + 10942.1Z$$
$$+ 679ZD^* = 0$$

$$-339.5D^{*2} - 7544.6D^* + 10942.1Z$$
$$+ 679ZD^* - 33866.9 = 0$$

& solve for Z:

$$Z(679D^* + 10942.1) = 33866.9 + 7544.6D^* + 339.5D^{*2}$$

Eqn A

$$\boxed{Z = \frac{33866.9 + 7544.6D^* + 339.5D^{*2}}{679D^* + 10942.1}}$$

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this is one eqn in 2 unknowns; need 2nd eqn:

$$\sum M_{\text{BOTTOM OF SHEET}} = 0$$

$$3775.2\left(D^* + 16' + \frac{11'}{3}\right) + 10982.4\left(D^* + \frac{16'}{2}\right)$$

$$+ 686.4 D^*\left(D^*/2\right) + 688.5\left(D^* + 16' + \frac{3'}{2}\right)$$

$$+ 390.9\left(\underbrace{D^* + 12.86}_{Y} + \frac{2}{3}\left(\underbrace{3.14'}_{X'}\right)\right)$$

$$- 3865\left(D^* + 12.86\right)\left(\frac{D^* + 12.86}{3}\right)$$

$$- 339.5 D^*\left(D^* + 12.86\right)\left(\frac{D^* + 12.86}{3}\right)$$

$$+ 10942.1 z\left(\frac{z}{3}\right) + 679 z D^*\left(\frac{z}{3}\right) = 0$$

$$\Downarrow$$

Simplifying

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$$3775.2D^* + 60403.2 + 13842.4 + 10982.4D^*$$
$$+ 87859.2 + 343.2D^{*2} + 688.5D^* + 11016$$
$$+ 1032.8 + 390.9D^* + 5027 + 818.2$$
$$-(3865D^* + 49703.9)\left(\frac{D^* + 12.86}{3}\right)$$
$$-(337.5D^{*2} + 4366D^*)\left(\frac{D^* + 12.86}{3}\right)$$
$$+ 3647.4Z^2 + 226.3D^*Z^2 = 0$$

$$\boxed{\begin{aligned}&179{,}998.8 + 15837D^* + 343.2D^{*2}\\ &-(3865D^* + 49703.9)\left(\frac{D^* + 12.86}{3}\right)\\ &-(339.5D^{*2} + 4366D^*)\left(\frac{D^* + 12.86}{3}\right)\\ &+ 3647.4Z^2 + 226.3D^*Z^2 = 0\end{aligned}}$$

to eqn. (B)

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Eqns (A) + (B) are $f(z, D^*)$ only + can be solved simultaneously:

Trial #1: Assume $D^* = 2.0'$

$z = 4.09'$ from Eqn (A)

LHS Eqn (B) $= -52837.6 \; (\neq 0)$

+ since negative, assumed D^* is too deep

Trial #2: Assume $D^* = 1.0'$

$z = 3.59'$ from Eqn (A)

LHS Eqn (B) $= -23124.1 \; (\neq 0)$

+ since negative, assumed D^* is too deep

Trial #3: Assume $D^* = 0'$

$z = 3.10$ from Eqn (A)

LHS Eqn (B) $= +1986.3 \approx 0$

$\therefore D^* = 0$

SOLN!

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∴ $D^* = 0'$ analysis shows that sheeting driven to top of stratum ④ will satisfy statics of this problem.

i.e. $D_{CALC} = 4' + 5' + 7' + \cancel{D^*}^{\,0}$

$= 16'$

$D_{DESIGN} = (1.2)(16') = 19.2'$ → use 19'

FS for Temporary Construction

∴ $L = 19' + 3' + 8' = 30'$

min reqd length of sheets

Determine elevation of zero shear; Assume that this occurs within stratum ③ @ distance "a" below top of stratum (see pg 2):

$\frac{1}{2}(686.4)(11') + 686.4(9') + 686.4(a)$

X'

$+ 3'\left(\frac{210+249}{2}\right) + \frac{1}{2}(249)(3.14')$

$- \frac{1}{2}(4' - 3.14')\left[(93-62.4)(3.01-.42)(.86')\right] - \ldots$ see next pg

pcf

.86

68.2 psf

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984.8

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$$-(5')\left\{\frac{68.2\,\text{psf} + \left[(108\overset{\text{pcf}}{-}62.4)(4.36-.34)(5') + 68.2\right]}{2}\right\}$$

$$-(a)\left\{\frac{984.8 + \left[(120\overset{\text{pcf}}{-}62.4)(7.66-.26)(a) + 984.8\right]}{2}\right\}$$

3376.0

simplify

$$3775.2 + 6177.6 + \cancel{686.4a} + 688.5$$
$$+ 390.9 - 29.3 - 2632.4$$
$$- \cancel{984.8a} - \cancel{213.1}\,a^2 = 0$$

$$-213.1a^2 - 298.4a + 8370.5 = 0$$

$$213.1a^2 + 298.4a - 8370.5 = 0$$

Quad eqn with $a = +213.1$, $b = +298.4$, $c = -8370.5$

$$X_{1,2} = \frac{-b \pm \sqrt{b^2-4ac}}{2a} = \frac{-298.4 \pm \sqrt{(298.4)^2 - 4(213.1)(-8370.5)}}{2(213.1)}$$

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$$a = x \left(\begin{matrix}\text{positive}\\ \text{root}\end{matrix}\right) = 5.61' < 7' \quad \text{OK}$$

i.e. Elev. of zero shear occurs in stratum 3 (see pg 2)

Then $\sum M_{\substack{\text{elev}\\ \text{of zero}\\ \text{shear}}} = M_{max}$

$$\text{i.e. } M_{max} = 3775.2\left(5.61' + 9' + 11'/3\right)$$

$$+ 6177.6\left(5.61' + 9'/2\right) + 686.4\left(5.61'\right)\left(5.61'/2\right)$$

$$+ 688.5\left(5.61' + 9' + 3'/2\right) + 390.9\left(5.61' + 5' + .86' + \tfrac{2}{3}(3.14')\right)$$

$$- 29.3\left(5.61' + 5' + .86'/3\right) - 68.2\left(5'\right)\left(5.61' + 5'/2\right)$$

$$- \frac{1}{2}\left(5'\right)\left(\underset{\text{psf}}{984.8 - 68.2}\right)\left(5.61' + 5'/3\right) - 984.8\left(5.61'\right)\left(\frac{5.61'}{2}\right)$$

$$- \frac{1}{2}\left(5.61'\right)\left(\underset{\text{psf}}{3376 - 984.8}\right)\left(\frac{5.61'}{3}\right)$$

$$= 110{,}849.4 \ \frac{\text{ft-lb}}{\text{ft}}$$

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$$\therefore \quad S_{REQD} \geq \frac{\left(110849.4 \ \frac{ft\,lb}{ft}\right)\left(12 \ in/ft\right)}{.65\,(50,000 \ psi)}$$

$$S_{REQD} \geq 40.93 \ in^3/ft$$

AZ-26 reqd!

$(S_{ACT} = 48.4 \ in^3/ft)$

Using a 10% permissible overstress factor (i.e. 1.10 in denominator of above eqn) for temporary construction:

$$S_{REQD} \geq \frac{\left(110,849.4 \ \frac{ft\,lb}{ft}\right)\left(12 \ in/ft\right)}{1.10\,(.65)(50,000 \ psi)}$$

$$= 37.21 \ in^3/ft$$

AZ-26 still reqd!

$(S_{ACT}\ (AZ\text{-}18) = 33.5 \ in^3/ft$ NG$)$

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Also, sheets must be manufactured from ASTM A-572 (Grade 50) [i.e. 50 ksi yield strength] steel, & must have hot rolled interlocks for adequate water inflow control. Purchased length should be 30'.

W L Deutsch, Ph.D., P.E.
3/11/02

APPENDIX A

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Correlation of γ_T to N

A.) Coarse Grained Soils

- see pg 2 Table
- From this Table:

N	γ_T
4	$(85 + 102.5)/2 = 94$ pcf
10	$(102.5 + 120)/2 = 111$ pcf
30	$(120 + 130)/2 = 125$ pcf
50	$(130 + 140)/2 = 135$ pcf

- See plot of this data on pg 3

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Table 3-3. Empirical values for ϕ, D_r, and unit weight of granular soils based on the standard penetration number with corrections for depth and for fine saturated sands

Description	Very loose	Loose	Medium	Dense	Very dense
Relative density D_r*	0 — 0.15	0.35	0.65	0.85	1.00
Standard penetration no. N	4	10	30	50	
Approx. angle of internal friction ϕ°†	25°–30° — 27–32°	30–35°	35–40°	38–43°.	
Approx. range of moist unit weight, (γ) pcf (kN/m³)	85 AVE 70–100‡ (11–16)	102.5 AVE 90–115 (14–18)	120 AVE 110–130 (17–20)	130 AVE 120 ~~110~~–140 (17–22)	140 AVE 130–150 (20–23)

* USBR [Gibbs and Holtz (1957)].

† After Meyerhof (1956). $\phi = 25 + 25D_r$ with more than 5 percent fines and $\phi = 30 + 25D_r$ with less than 5 percent fines. Use larger values for granular material with 5 percent or less fine sand and silt.

‡ It should be noted that excavated material or material dumped from a truck will weigh 70 to 90 pcf. ← NFO condition
Material must be quite dense and hard to weigh much over 130 pcf. Values of 105 to 115 pcf for nonsaturated soils are common.

Ref: Bowles, 2nd ed.; "Fnd. Analysis + Design"

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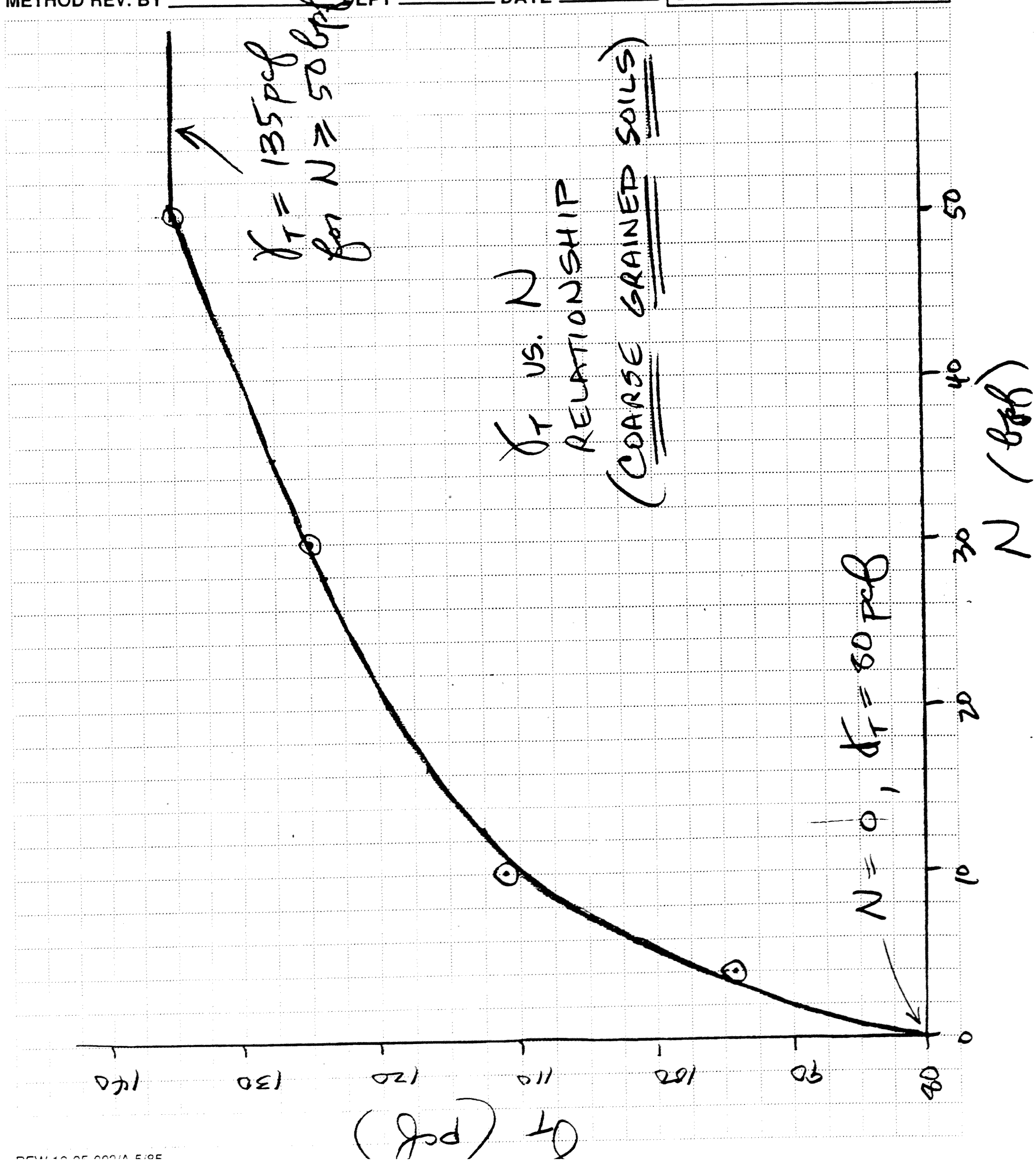
DEPT ________ DATE ________



APPENDIX B

(REF: USS STEEL SHEET PILING DESIGN MANUAL)

B-1

DESIGN OF SHEET PILE RETAINING WALLS

GENERAL CONSIDERATIONS

The design of sheet pile retaining walls requires several successive operations: (a) evaluation of the forces and lateral pressures that act on the wall, (b) determination of the required depth of piling penetration, (c) computation of the maximum bending moments in the piling, (d) computation of the stresses in the wall and selection of the appropriate piling section and (e) the design of the waling and anchorage system. Before these operations can be initiated, however, certain preliminary information must be obtained. In particular, the controlling dimensions must be set. These include the elevation of the top of the wall, the elevation of the ground surface in front of the wall (commonly called the dredge line), the maximum water level, the mean tide level or normal pool elevation and the low water level. A topographical survey of the area is also helpful.

Earth pressure theories have developed to the point where it is possible to obtain reliable estimates of the forces on sheet pile walls exerted by homogeneous layers of soil with known physical constants. The uncertainties involved in the design of sheet pile structures no longer result from an inadequate knowledge of the fundamentals involved. They are caused by the fact that the structure of natural soil deposits is usually quite complex, whereas the theories of bulkhead design inevitably presuppose homogeneous materials. Because of these conditions, it is essential that a subsurface investigation be performed with exploratory borings and laboratory tests of representative samples. On this basis, a soil profile can be drawn and the engineering properties of the different soil strata can be accurately determined. These properties should reflect the field conditions under which the wall is expected to operate. Only after these preliminary steps are taken should the final design be undertaken.

There are two basic types of steel sheet pile walls: cantilevered walls and anchored walls. The design of each type for various subsurface conditions will be discussed in the following sections.

CANTILEVER WALLS

In the case of a cantilevered wall, sheet piling is driven to a sufficient depth into the ground to become fixed as a vertical cantilever in resisting the lateral active earth pressure. This type of wall is suitable for moderate height. Walls designed as cantilevers usually undergo large lateral deflections and are readily affected by scour and erosion in front of the wall. Since the lateral support for a cantilevered wall comes from passive pressure exerted on the embedded portion, penetration depths can be quite high, resulting is excessive stresses and severe yield. Therefore, cantilevered walls using steel sheet piling are restricted to a maximum height of approximately 15 feet.

Earth pressure against a cantilevered wall is illustrated in Figure 14. When the lateral active pressure (P) is applied to the top of the wall, the piling rotates about the pivot point, b, mobilizing passive pressure above and below the pivot point. The term (p_p-p_a) is the net passive pressure, p_p, minus the active pressure, p_a. (Since both are exerting pressure upon the wall.)

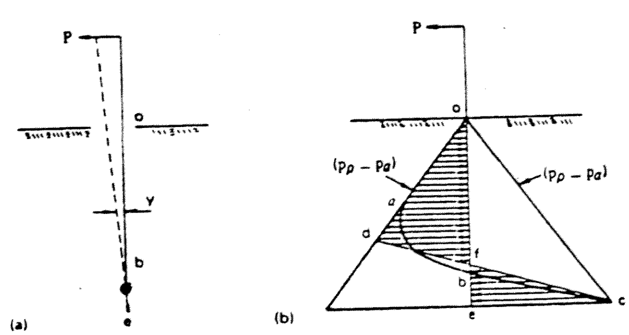


Fig. 14 — Earth pressure on cantilever sheet piling (after Teng[1])

At point b the piling does not move and would be subjected to equal and opposite at-rest earth pressures with a net pressure equal to zero. The resulting earth pressure is represented by the diagram oabc. For the purpose of design, the curve abc is replaced by a straight line dc. The point d is located so as to make the sheet piling in a state of static equilibrium. Although the assumed pressure distribution is in error, it is sufficient for design purposes.

The distrubution of earth pressure is different for sheet piling in granular soils and sheet piling in cohesive soils. Also, the pressure distribution in clays in likely to change with time. Therefore, the design procedures for steel sheet piling in both types of soils are discussed separetely.

Cantilever Sheet Piling in Granular Soils – A cantilevered sheet pile wall may be designed in accordance with the principles and assumptions just discussed or by an approximate method based on further simplifying assumptions shown in Figure 15.

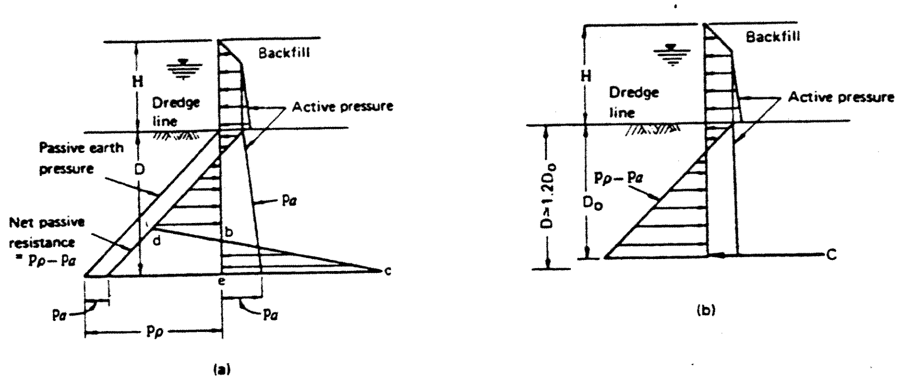


Fig. 15 – Design of cantilever sheet piling in granular soils: (a) conventional method; (b) simplified method. (after Teng[1])

For cases of two or more layers of soil, the earth pressure distributions would be somewhat different due to the different soil properties; however, the design concept is exactly the same. Lateral pressures should be calculated using the curved failure surface (log spiral) method as shown in Figure 5 (a). ← see pg 3A

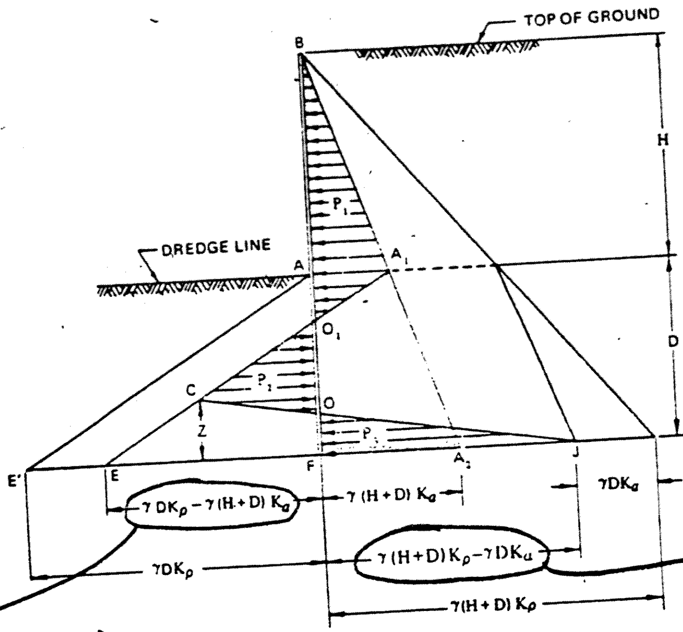
Conventional Method – The conventional design procedure for granular soils is as follows:

1. Assume a trial depth of penetration, D. This may be estimated from the following approximate correlation.

Standard Penetration Resistance, *N* Blows/Foot	Relative Density of Soil, D_d	Depth of Penetration* D
0-4	Very loose	2.0 H
5-10	Loose	1.5 H
11-30	Medium dense	1.25 H
31-50	Dense	1.0 H
+50	Very dense	0.75 H

*H = height of piling above dredge line.

2. Determine the active and passive lateral pressures using appropriate coefficients of lateral earth pressure. If the Coulomb method is used, it should be used conservatively for the passive case. The resulting earth pressure diagram for a homogeneous granular soil is shown in Figure 16 where the active and passive pressures are overlain to pictorially describe the resulting soil reactions.



P(H+D) - A(D)

P(D) - A(H+D) Fig. 16 – Resultant earth-pressure diagram

3. Satisfy the requirements of static equilibrium: the sum of the forces in the horizontal direction must be zero and the sum of the moments about any point must be zero. The sum of the horizontal forces may be written in terms of pressure areas:

$$\Delta(E\overrightarrow{A_1A_2}) - \Delta(\overleftarrow{FBA_2}) - \Delta(\overleftarrow{ECJ}) = 0$$

Solve the above equation for the distance, Z. For a uniform granular soil,

$$Z = \frac{K_p D^2 - K_a (H+D)^2}{(K_p - K_a)\ (H+2D)}$$

Take moments about the point F and check to determine if the sum of the moments is equal to zero, as it must be. Readjust the depth of penetration, D, and repeat until convergence is reached; i.e., the sum of the moments about F is zero.

4. Add 20 to 40 percent to the calculated depth of penetration. This will give a safety factor of approximately 1.5 to 2.0. An alternate and more desirable method is the use of a reduced value of the passive earth pressure coefficient for design. The maximum allowable earth pressure should be limited to 50 to 75 percent to the ultimate passive resistance.

5. Compute the maximum bending moment, which occurs at the point of zero shear, prior to increasing the depth by 20 to 40 percent.

A rough estimate of the lateral displacement may be obtained by considering the wall to be rigidly held at an embedment of ½D and subjected to a triangular load distribution approximating the actual applied active loading. The displacement at any distance y from the top of the pile is then given by the following expression:

$$\Delta_\ell = \frac{Pt}{60EI\ell^2} \cdot (y^5 - 5\ell^4 y + 4\ell^5)$$

SCENARIO II

SHEET PILE DESIGN
SHEETING LINE ALONG CENTERLINE OF RIVER
(WORST-CASE SCENARIO [5-FT CUT])

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MATH CHECK BY SW DEPT 1382 DATE 3/15/02
METHOD REV. BY DEPT DATE DEPT DATE

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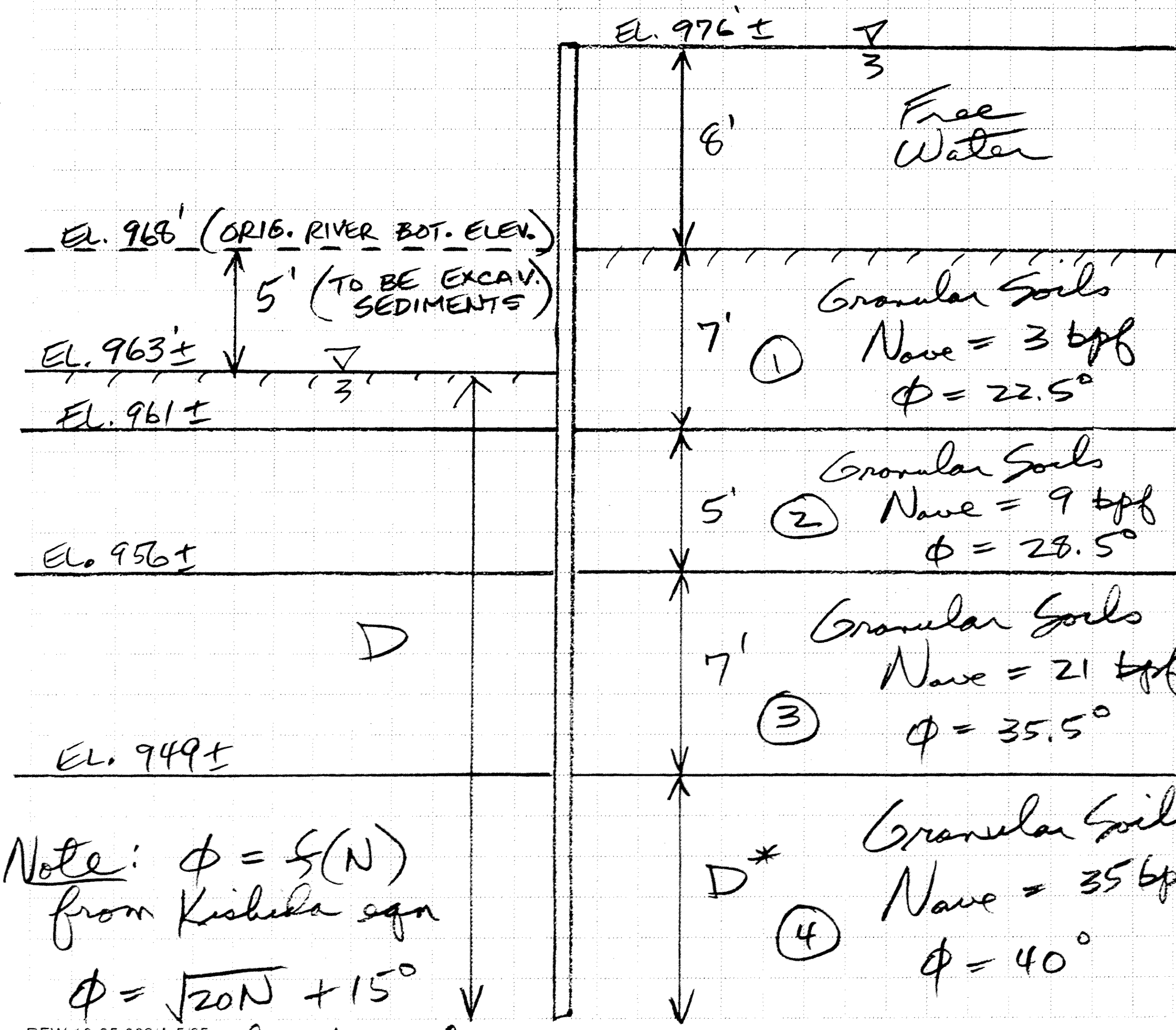
SCENARIO II
SHEET PILE DESIGN
SHEETING LINE ALONG ℄ RIVER
(WORST CASE SCENARIO (5' CUT))

Based on relevant borings within river channel and along river bank, the selected design cross section is:



Note: $\phi = f(N)$ from Kishida eqn

$$\phi = \sqrt{20N} + 15^\circ$$

for $\phi < 40^\circ$

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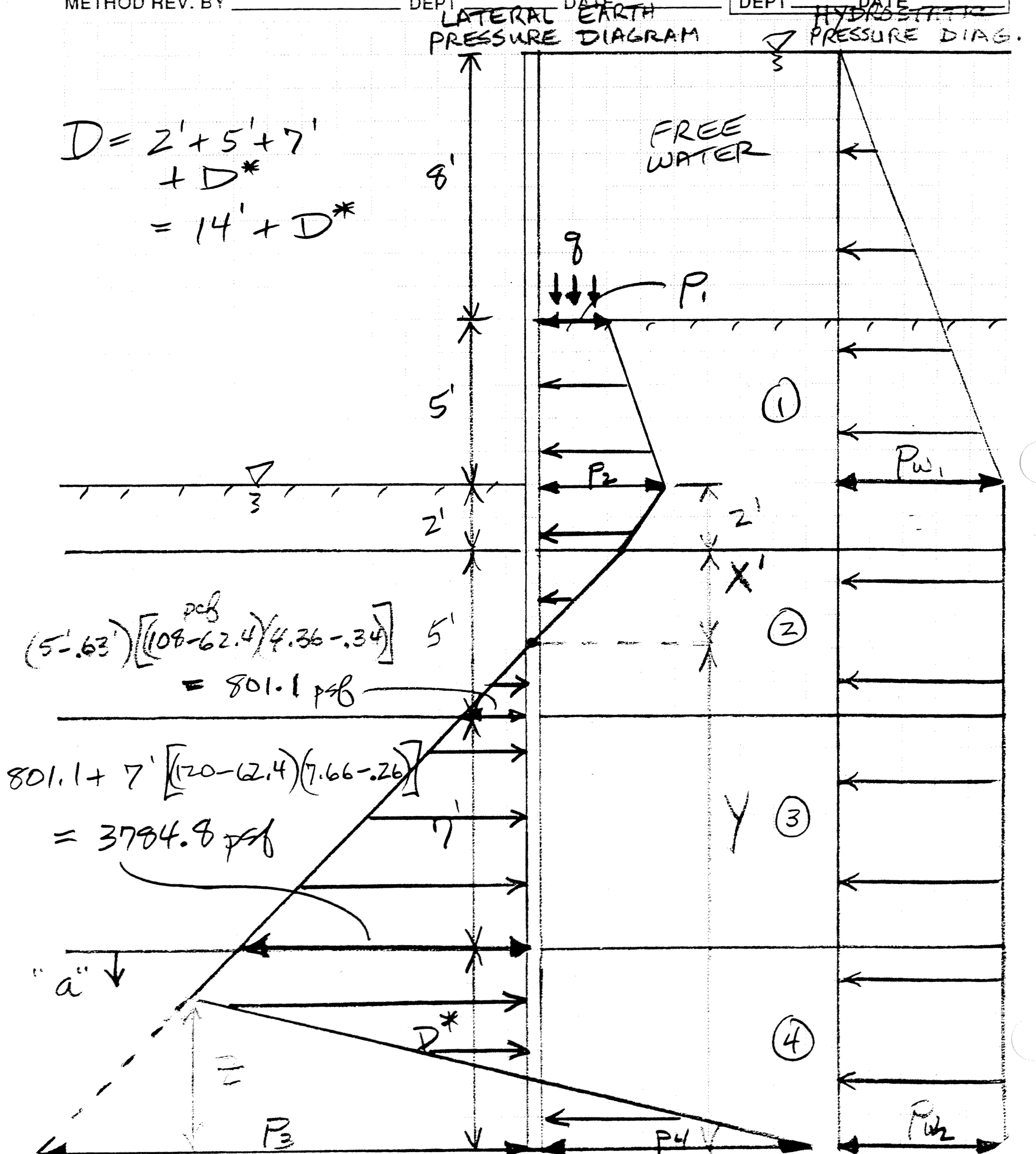

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$P_{w_1} = 13' \times 62.4 \text{ pcf} = 811.2 \text{ psf}$

$P_{w_2} = P_{w_1} = 811.2 \text{ psf}$

K_A & K_P VALUES: (see Figure on pg 3A from Pile Buck Sheet Pile Design Manual)

1.) Stratum 1

$K_A: \quad \phi = 22.5°$

$\beta/\phi = 0°$

$\delta/\phi = .5$ (ASSUMED)

$\boxed{K_A = .42}$

$K_P: \quad \phi = 22.5°$

$\beta/\phi = 0° \rightarrow K_{P_u} = 3.6$

$\delta/\phi = .5 \rightarrow R = .835$

$K_P = 3.6(.835) = 3.01$

$\boxed{K_P = 3.01}$

3A

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

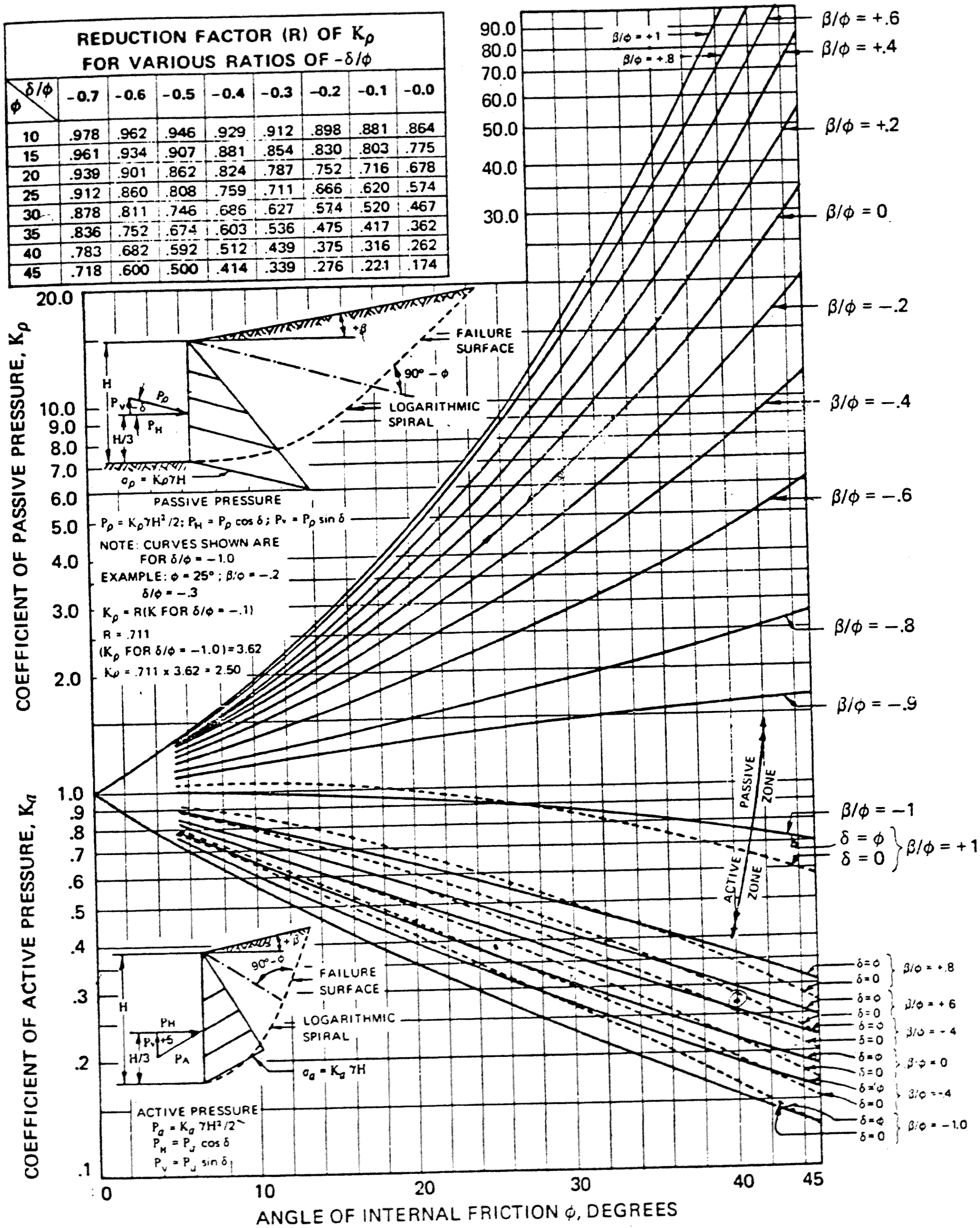

Fig. 5(a) – Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[21])

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2.) Stratum 2

K_A: $\phi = 28.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$\boxed{K_A = .34}$

K_P: $\phi = 28.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$K_{P_u} = 5.7$ $R = .765$

$\boxed{K_P = 5.7(.765) = 4.36}$

3.) Stratum 3

K_A: $\phi = 35.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$\boxed{K_A = .26}$

K_P: $\phi = 35.5°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$K_{P_u} = 11.5$ $R = .666$

$\boxed{K_P = 11.5(.666) = 7.66}$

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4.) Stratum 4

K_A: $\phi = 40°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$\boxed{K_A = .22}$

K_P: $\phi = 40°$

$\beta/\phi = 0°$ $\delta/\phi = .5$

$K_{Pu} = 18.0$ $R = .592$

$K_P = 18.0\,(.592) = 10.65$

$\therefore \boxed{K_P = 10.65}$

Unit Weights (γ_{SAT}):

$\gamma_{SAT} = f(N)$

– see Appendix A; From this data

Soil Stratum	γ_{SAT} (pcf)
1	93
2	108
3	120
4	127.5

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Then $P_1 = q K_A = [8' \times 62.4 \text{pcf}](.42) = 210 \text{ psf}$

surcharge load due to wt of 8' of free water

See pg 2 Figure

$P_2 = P_1 + (93 - 62.4 \text{ pcf})(5')(.42) = 274.3 \text{psf}$

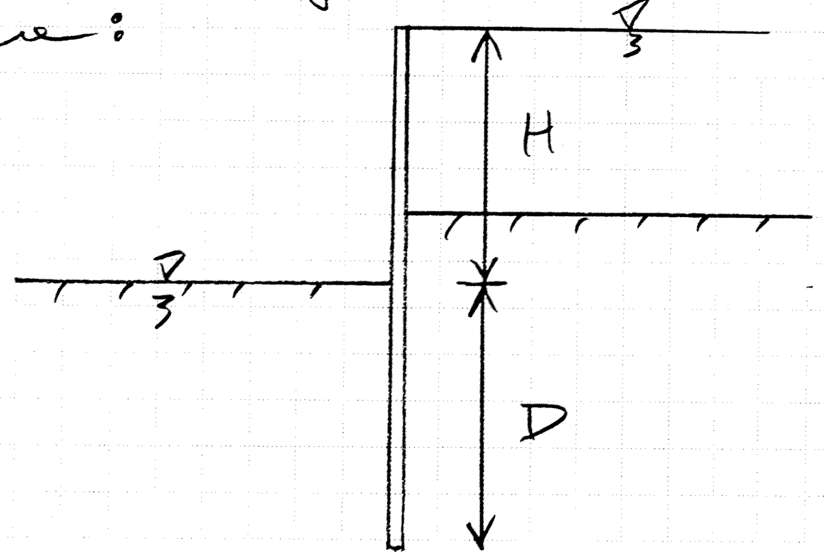
Calculate:

see pg 2 $\rightarrow P_3 = P(D) - A(H+D) \rightsquigarrow$ see App B, pg 3-3

Passive LEP calc based on all strata within Distance D to bottom of sheeting

Active LEP calc based on all strata within Distance (H + D) to bottom of sheeting

where:



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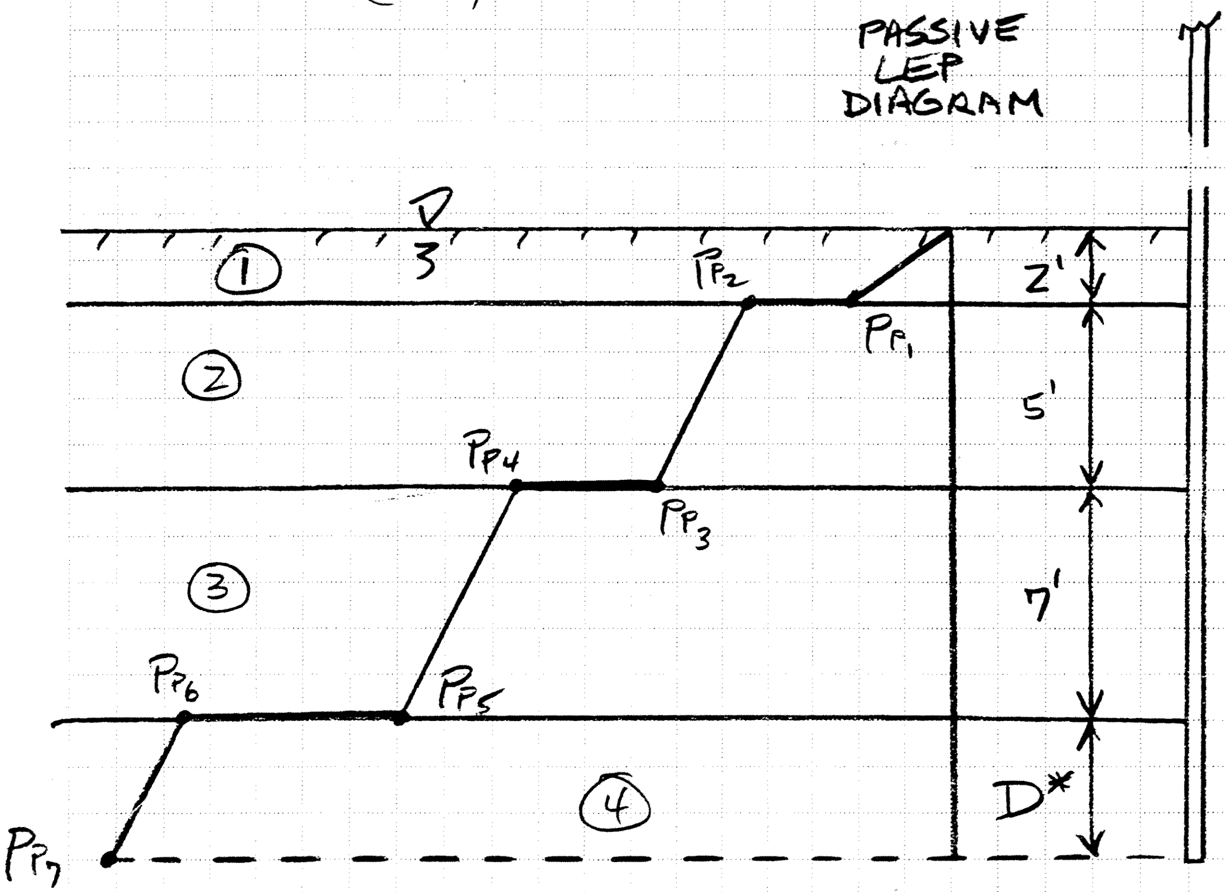
Then P(D) is calculated as:



$$P_{P_1} = (93_{pcf} - 62.4)(2')(3.01) = 184.2 \text{ psf}$$

$$P_{P_2} = [(93 - 62.4)(2')](4.36) = 266.8 \text{ psf}$$

$$P_{P_3} = 266.8 + (108_{pcf} - 62.4)(5')(4.36) = 1260.9 \text{ psf}$$

$$P_{P_4} = [(93_{pcf} - 62.4)(2') + (108_{pcf} - 62.4)(5')](7.66) = 2215.3 \text{ psf}$$

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$$P_{P_5} = 2215.3 + (120 - 62.4)(7')(7.66)$$
$$= 5303.8 \text{ psf}$$

$$P_{P_6} = \Big[(\underset{\text{pcf}}{93 - 62.4})(2') + (\underset{\text{pcf}}{108 - 62.4})(5')$$
$$+ (\underset{\text{pcf}}{120 - 62.4})(7') \Big] (10.65)$$
$$= 7374.1 \text{ psf}$$

$$P_{P_7} = 7374.1 + (127.5 - 62.4) D^* (10.65)$$
$$= 7374.1 + 693.3 D^*$$

Also, $A(H+D)$ is calculated as:

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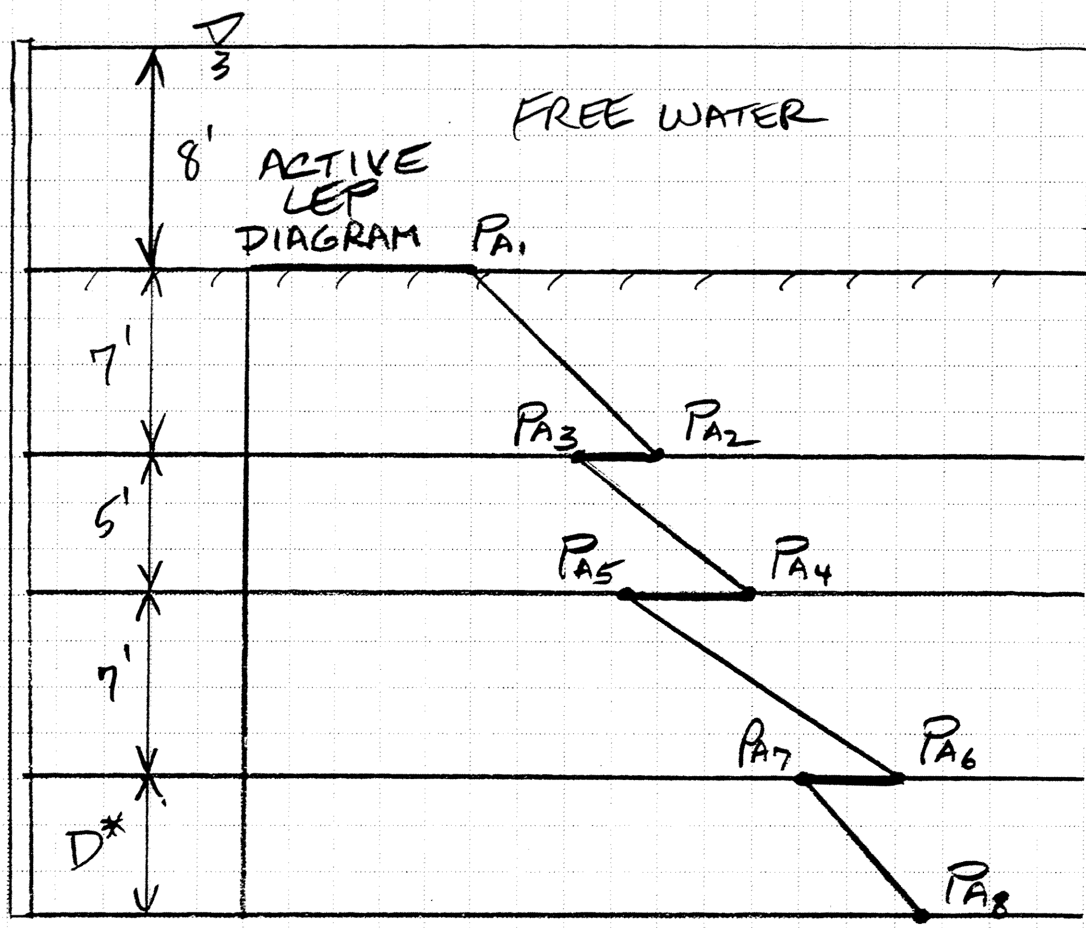

$$P_{A1} = qK_A = (8' \times 62.4\ \text{pcf})(.42) = 210\ \text{psf}$$

$$P_{A2} = 210 + (93 - 62.4\ \text{pcf})(7')(.42) = 300\ \text{psf}$$

$$P_{A3} = [8 \times 62.4 + (93 - 62.4)(7')](.34) = 242.6\ \text{psf}$$

$$P_{A4} = 242.6 + (108 - 62.4)(5')(.34) = 320.1\ \text{psf}$$

$$P_{A5} = [8 \times 62.4 + (93 - 62.4)(7) + (108 - 62.4)(5)](.26) = 244.8\ \text{psf}$$

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$$P_{A_6} = 244.8 + (120-62.4)(7')(.26) = 349.6 \text{ psf}$$

$$P_{A_7} = \left[8 \times 62.4 + (93-62.4)(7) + (108-62.4)(5) + (120-62.4)(7)\right](.22) = 295.8 \text{ psf}$$

$$P_{A_8} = 295.8 + (127.5-62.4)(D^*)(.22) = 295.8 + 14.3D^*$$

$$\alpha \quad P_3 = P(D) - A(H+D)$$

See pg 2

$$= P_{P_7} - P_{A_8} = \left[7374.1 + 693.3D^*\right] - \left[295.8 + 14.3D^*\right]$$

See pgs 7 + 8   See pgs 9 + above

$$\therefore \boxed{P_3 = 7078.3 + 679 D^*}$$

Calculate:

See pg 2

$$P_4 = P(H+D) - A(D)$$

see App B pg B-3

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P(H+D) is calculated as follows with reference to Fig on pg 9 where P_A values are replaced by P_P values calc. using K_p values:

$$P_{P_1} = qK_p = (8 \times 62.4)(3.01) = 1502.6 \text{ psf}$$

$$P_{P_2} = 1502.6 + (93-62.4)(7)(3.01) = 2147.3 \text{ psf}$$

$$P_{P_3} = \left[8 \times 62.4 + (93-62.4)(7')\right](4.36) = 3110.4 \text{ psf}$$

$$P_{P_4} = 3110.4 + (108-62.4)(5')(4.36) = 4104.5 \text{ psf}$$

$$P_{P_5} = \left[8 \times 62.4 + (93-62.4)(7) + (108-62.4)(5)\right](7.66)$$
$$= 7211.1 \text{ psf}$$

$$P_{P_6} = 7211.1 + (120-62.4)(7)(7.66)$$
$$= 10299.6 \text{ psf}$$

$$P_{P_7} = \left[8 \times 62.4 + (93-62.4)(7) + (108-62.4)(5) + (120-62.4)(7)\right](10.65)$$
$$= 14320.0 \text{ psf}$$

$$P_{P_8} = 14320 + (127.5-62.4)(D^*)(10.65)$$
$$= 14320 + 693.3D^*$$

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A (D) is calculated as follows with reference to Fig on pg 7 where P_P values are replaced by P_A values calc. using K_A values:

$$P_{A_1} = (93-62.4)(2')(.42) = 25.7 \text{ psf}$$

$$P_{A_2} = [(93-62.4)(2')](.34) = 20.8 \text{ psf}$$

$$P_{A_3} = 20.8 + (108-62.4)(5')(.34) = 98.3 \text{ psf}$$

$$P_{A_4} = [(93-62.4)(2') + (108-62.4)(5')](.26)$$
$$= 75.2 \text{ psf}$$

$$P_{A_5} = 75.2 + (120-62.4)(7')(.26) = 180.0 \text{ psf}$$

$$P_{A_6} = [(93-62.4)(2) + (108-62.4)(5) + (120-62.4)(7)](.22) = 152.3 \text{ psf}$$

$$P_{A_7} = 152.3 + (127.5-62.4)(D^*)(.22)$$
$$= 152.3 + 14.3 D^*$$

α $P_4 = P(H+D) - A(D) = P_{P_8} - P_{A_7}$

see pg 2 see pg 11 see [illegible]

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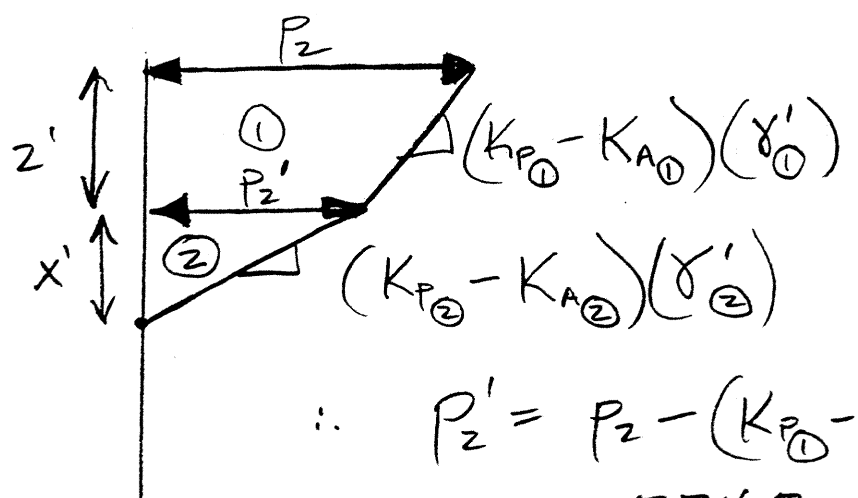
$$\therefore \quad P_4 = \left[14320 + 693.3D^*\right] - \left[152.3 + 14.3D^*\right]$$

$$\therefore \quad \boxed{P_4 = 14167.7 + 679\,D^*}$$

Now determine distance x' (see pg 2) as follows:

– assume x' lies within stratum 2



$$\therefore \quad P_2' = P_2 - (K_{P_1} - K_{A_1})(\gamma'_1)(2')$$

$$= 274.3\,\text{psf} - (3.01 - .42)(\underset{\text{pcf}}{93 - 62.4})(2')$$

$$= 115.8\,\text{psf}$$

Then $\quad P_2' - (K_{P_2} - K_{A_2})(\gamma'_2)\,x' = 0\,\text{psf}$

$$115.8\,\text{psf} - (4.36 - .34)(\underset{\text{pcf}}{108 - 62.4})\,x' = 0$$

$$x' = .63' < 5' \rightarrow$$

$\therefore$ x' lies within stratum 2; assumption OK

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Then $\Sigma F_H = 0$ $\left(\overset{+}{\leftarrow}\right)$

$$\frac{1}{2}(811.2\,psf)(13') + (811.2\,psf)(2'+5'+7') + (811.2\,psf)D^*$$

$$+\ 5'\left(\frac{210 + 274.3}{2}\right)^{psf} + \frac{1}{2}(2')(274.3 - 115.8)_{psf} + (115.8\,psf)(2$$

$$+\ \frac{1}{2}(.63')(115.8\,psf) - \frac{1}{2}\left[7078.3 + 679\,D^*\right](Y)$$

see pg 2

$$+\ \frac{1}{2}(Z)\left[(7078.3 + 679\,D^*) + (14167.7 + 679\,D^*)\right]$$

see pg 2

$$= 0$$

$$\cancel{5272.8} + \cancel{11356.8} + 811.2\,D^* + \cancel{1210.8} + \cancel{158.5} + \cancel{231.6}$$

$$+ \cancel{36.5} - 3539.2\,Y - 339.5\,D^*Y$$

$$+ 10623\,Z + 679\,Z\,D^* = 0$$

Eqn is $f(D^*, Z \,\&\, Y)$; eliminate one variable noting that (see pg 2):

$$D^* + 7' + 5' + 2' = 2' + \underbrace{.63'}_{x'} + Y$$

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$$\therefore \quad Y = D^* + 11.37'$$

Sub in above eqn & simplify:

$$\cancel{18267} + \cancel{811.2D^*} - 3539.2\cancel{(D^* + 11.37')}$$
$$- 339.5D^*\cancel{(D^* + 11.37')} + \cancel{10{,}623Z} + \cancel{679ZD^*} = 0$$

$$-339.5D^{*2} - 6588.1D^* + 10{,}623Z + 679ZD^* - 21{,}973.7 = 0$$

solve for Z

$$Z(10623 + 679D^*) = 339.5D^{*2} + 6588.1D^* + 21973.7$$

Eqn A

$$\boxed{Z = \frac{339.5D^{*2} + 6588.1D^* + 21973.7}{10623 + 679D^*}}$$

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This is one eqn in 2 unknowns; need 2nd eqn:

$$\Sigma M_{\substack{BOTTOM\\SHEETS}} = 0$$

$$5272.8\left(D^* + 14' + 13'/3\right) + 11356.8\left(D^* + \frac{14'}{2}\right)$$

$$+ 811.2\,D^*\left(\frac{D^*}{2}\right) + (1210.8)\left(D^* + 14' + \frac{5'}{2}\right)$$

$$+ 158.5\left(D^* + 12' + \frac{2}{3}(2')\right) + 231.6\left(D^* + 12 + \frac{2'}{2}\right)$$

$$+ 36.5\left(D^* + 7' + (5' - .63') + .63'\left(\frac{2}{3}\right)\right)$$

$$- 3539.2\left(D^* + 11.37'\right)\left(\frac{D^* + 11.37}{3}\right)$$

$$- 339.5\,D^*\left(D^* + 11.37\right)\left(\frac{D^* + 11.37'}{3}\right)$$

$$+ 10623\,z\left(\frac{z}{3}\right) + 6792\,D^*\left(\frac{z}{3}\right) = 0$$

Simplify

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$$5272.8D^* + 73819.2 + 22848.8 + 11356.8D^*$$
$$+ 79497.6 + 405.6D^{*2} + 1210.8D^* + 16951.2$$
$$+ 3027 + 158.8D^* + 1902 + 211.3 + 231.6D^*$$
$$+ 2779.2 + 231.6 + 36.5D^* + 255.5 + 159.5$$
$$+ 15.3 - \left(3539.2D^* + 40{,}240.7\right)\left(\frac{D^* + 11.37}{3}\right)$$
$$- \left(339.5D^{*2} + 3860.1D^*\right)\left(\frac{D^* + 11.37}{3}\right)$$
$$+ 3541Z^2 + 226.3Z^2D^* = 0$$

$$\downarrow$$

$$\boxed{\begin{aligned} &201{,}698.2 + 18267D^* + 405.6D^{*2} \\ &- \left(3539.2D^* + 40{,}240.7\right)\left(\frac{D^* + 11.37}{3}\right) \\ &- \left(339.5D^{*2} + 3860.1D^*\right)\left(\frac{D^* + 11.37}{3}\right) \\ &+ 3541Z^2 + 226.3Z^2D^* = 0 \end{aligned}}$$

Eqn B

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Eqns (A) + (B) are $f(Z, D^*)$ only + can be solved simultaneously:

Trial #1: Assume $D^* = 2.0'$

$Z = 3.05' \longrightarrow$ Eqn (A)

LHS Eqn (B) $= +25661 \neq 0$

(NG)

+ since +, assumed D^* is too low

Trial #2: Assume $D^* = 3.0'$

$Z = 3.54' \longrightarrow$ Eqn (A)

LHS Eqn (B) $= -685.0 \approx 0$

OK, soln!

+ since slightly negative $D^* = 3.0'$ is slightly conservative

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$\therefore \quad D_{THEO} = D^{*} + 14'$ (see pg 2)

$D_{THEO} = 3' + 14 = 17'$

& recommended FS on D_{THEO} is $1.2 \rightarrow 1.4$; use 1.2 (Temporary Construction)

$\therefore \quad D_{THEO} = 17'(1.2) = 20.4'$

& $L = 20.4 + 5' + 8' = 33.4'$

- L: read length of piling
- 20.4: embedment depth below excavation bottom
- $5'$: depth excavation
- $8'$: max water lt. in sewer

$\therefore$ Use 35' long sheets & this will provide FS of:

Embedment of sheeting $= 35' - (5' + 8') = 22'$

& $FS = \dfrac{22'}{17'} = 1.29 > 1.2$

OK

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If it is desired to use the 30' long sheets that were sized for the 3' deep typical excavation depth (see separate set of calcs.):

Then: Embedment of Sheets $= 30' - (5 + 8') = 17'$

& $FS = \frac{17'}{17'} = 1.0$

i.e. Incipient Failure Condition @ which sheets would be expected to lean significantly toward excavated area as a result of insufficient embedment

Can only allow 30' sheets to be used for a 5' deep excavation of river bottom sediments if these localized excavations are:

1.) Only completed during low water surface elevation conditions in river.

2.) Localized deeper excavations are backfilled with controlled compacted fill immediately [illegible]

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Determine elevation of zero shear; Assume that this occurs within stratum ④ @ distance "a" below top of stratum (see pg 2):

$$\frac{1}{2}(811.2\,psf)(13') + (811.2\,psf)(2+5+7) + (811.2\,psf)(a)$$

$$+\ 5'\left(\frac{210 + 274.3}{2}\right)psf + \frac{1}{2}(2')(274.3 - 115.8)\,psf + (115.8\,psf)(2')$$

$$+\ \frac{1}{2}(.63')(115.8\,psf) - \frac{1}{2}(801.1\,psf)(\underbrace{5 - .63}_{4.37'})$$

see pg 2

$$-\ 7'\left(\frac{801.1 + 3784.8}{2}\right)psf - a\left(\frac{3784.8 + [3784.8 + a(127.5 - 62.4)(10.65 - ?)]}{2}\right)$$

$$= 0$$

$$5272.8 + 11356.8 + 811.2a + 1210.8 + 158.5 + 231.6$$

$$+\ 36.5 - 1750.4 - 16050 - a(3784.8 + 339.5a) = 0$$

$$466.6 + 811.2a - 3784.8a - 339.5a^2 = 0$$

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$$339.5a^2 + 2973.6 - 466.6 = 0$$

quadratic eqn with
$a = +339.5$
$b = +2973.6$
$c = -466.6$

$$\text{"a"} = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$\text{"a"} = \frac{-2973.6 \pm \sqrt{(2973.6)^2 - 4(339.5)(-466.6)}}{2(339.5)}$$

only + root is valid

$$\text{"a"} = .15' < 3.0' \longrightarrow \therefore \text{ soln OK}$$

D*

i.e. Elev of zero shear occurs .15' below the interface of stratum 3 & 4 (see pg 2).

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$$M_{max} = \Sigma M_{\text{elev of zero shear}}$$

$$5272.8\left(.15' + 7' + 5' + 2' + 13'/3\right) + 11356.8\left(.15' + 14'/2\right)$$

$$+ 811.2\left(.15'\right)\left(.15'/2\right) + 1210.8\left(.15' + 7' + 5' + 2' + 5'/2\right)$$

$$+ 158.5\left(.15' + 7' + 5' + \frac{2}{3}(2')\right) + 231.6\left(.15 + 7' + 5' + \frac{2'}{2}\right)$$

$$+ 36.5\left(.15' + 7' + (5' - .63') + \frac{2}{3}(.63')\right) - 1750.4\left(.15' + 7' + \frac{4.37'}{3}\right)$$

$$- \left[(801.1\ psf)(7')\right]\left(.15' + 7'/2\right) - \left[\frac{1}{2}(7')\left(\underset{psf}{3784.8} - 801.1\right)\right]\left(.15' + 7'/3\right)$$

$$- \left[(3784.8)(.15')\right]\left(.15'/2\right) - \left\{\frac{1}{2}(.15)\left[.15(127.5 - 62.4)(10.65 - .22)\right]\right\}\left(\ldots\right.$$

$$= 97458.9 + 81201.1 + 9.1 + 20159.8$$

$$+ 2137.1 + 3045.5 + 435.8 - 15065.1$$

$$- 20468.1 - 25933.3 - 42.6 - .4$$

$$= 142{,}937.8\ ft\text{-}lb/ft$$

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$$\therefore S_{REQD} \geq \frac{\left(142937.8 \frac{ft\text{-}lb}{ft}\right)\left(12 in/ft\right)}{.65\left(50{,}000 psi\right)}$$

$$S_{REQD} \geq 52.78$$

AZ-26 No Good

$(S_{ACT} = 48.4\ in^3/ft)$

But, can use a 10% permissible overstress factor (i.e. 1.10 in denominator of above eqn) for temporary construction:

$$S_{REQD} \geq \frac{\left(142937.8 \frac{ft\text{-}lb}{ft}\right)\left(12 in/ft\right)}{(1.10)(.65)(50{,}000 psi)}$$

$$S_{REQD} \geq 47.98\ in^3/ft$$

AZ-26 OK $\rightarrow S_{ACT} = 48.4\ in^3/ft$

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Also, sheets must be manufactured from ASTM A-572, Grade 50 [i.e. 50 ksi yield strength] steel, & must have hot rolled interlocks for adequate water inflow control. Purchased length should be 30'.

WL Deutsch
Ph.D., P.E.

3/11/02

PIPING ANALYSIS

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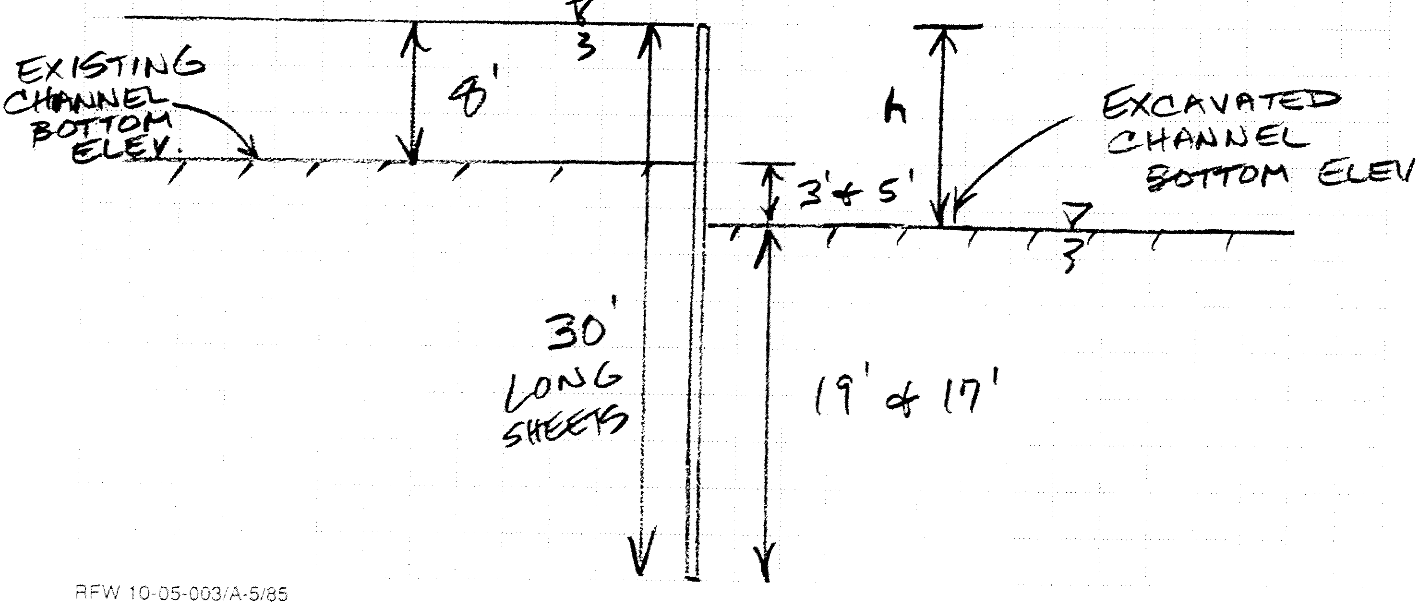
MANAGERS DESIGNERS/CONSULTANTS

CLIENT/SUBJECT GE - Pittsfield. SHEET 1 of ___ W.O. NO. ___
TASK DESCRIPTION Sheet Pile Design TASK NO. ___
PREPARED BY WCD DEPT 1274 DATE 3/5/02
MATH CHECK BY SW DEPT 1382 DATE 3/15/02
METHOD REV. BY ___ DEPT ___ DATE ___
APPROVED BY ___ DEPT ___ DATE ___

PIPING ANALYSIS

BACKGROUND: SHEET PILE DESIGN FOR CANTILEVERED SHEETING TO BE PLACED ALONG ALIGNMENT OF ℄ RIVER HAS BEEN COMPLETED AS DOCUMENTED IN A SEPARATE SET OF CALCULATIONS. A SEPARATE CONCERN RELATED TO THIS PROPOSED CONSTRUCTION IS THE UPWARD FLOW GRADIENT WHICH WILL BE GENERATED BY THE WATER HEAD DIFFERENTIAL CREATED WHEN THE TO BE EXCAVATED AREA IS DEWATERED. THIS ANALYSIS WAS COMPLETED TO DETERMINE IF PIPING OF THE GRANULAR SOILS AT THE BASE OF THE EXCAVATED AREA CAN OCCUR AS A RESULT OF THIS HEAD DIFFERENTIAL.

SHEET PILE DESIGN DETAILS ARE:



WESTON MANAGERS DESIGNERS/CONSULTANTS

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ANALYTICAL MODEL IS AS FOLLOWS:

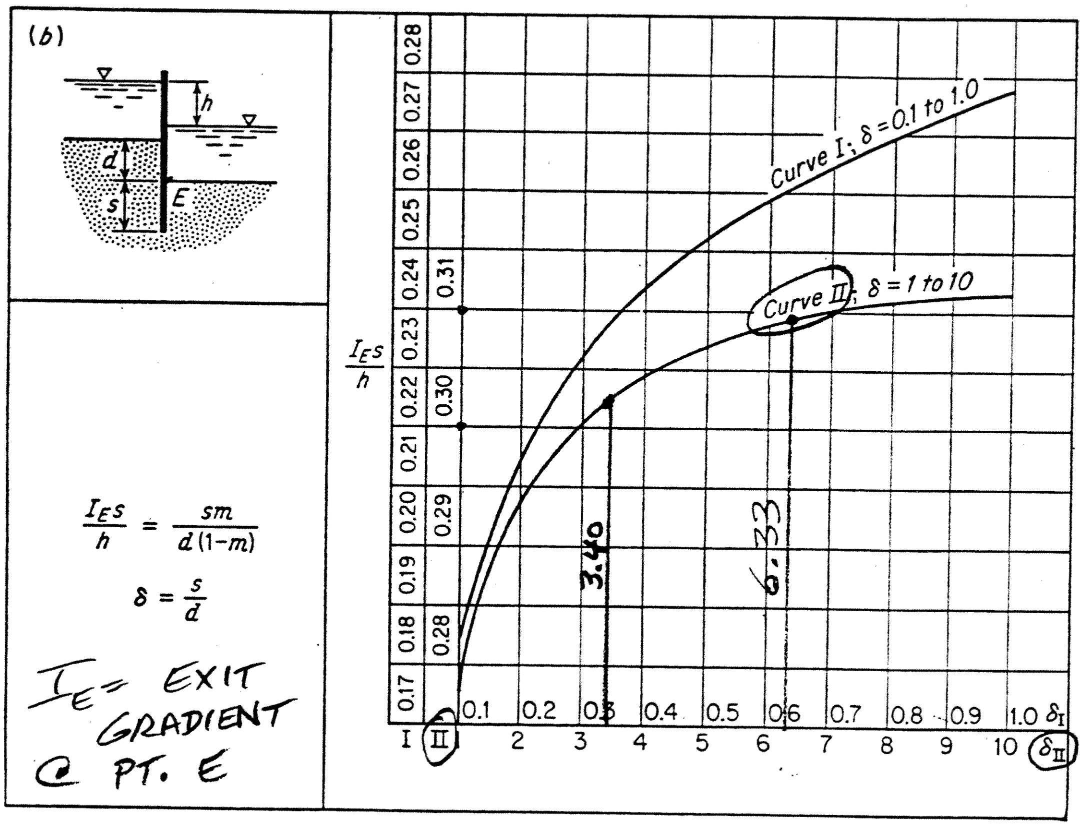

FIG. 5-9. (*After Khosla, Bose, and Taylor* [69].)

REF: HARR, M.E.; "GROUNDWATER & SEEPAGE"; 1962

FOR THIS DESIGN:

$s = 19' \& 17'$

$d = 3' \& 5'$

$h = 11' \& 13'$

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($d = 3'$ CASE): $\delta = \dfrac{s}{d} = \dfrac{19'}{3'} = 6.33$

USE CURVE II ON FIG. 5.9 ($\delta = 1 \rightarrow 10$)

$$\therefore \frac{I_E\, s}{h} = .3095$$

$$\therefore I_E = \frac{.3095\, h}{s} = \frac{.3095\,(11')}{19'}$$

$$I_E = .179$$

($d = 5'$ CASE): $\delta = \dfrac{s}{d} = \dfrac{17'}{5'} = 3.40$

USE CURVE II ON FIG. 5.9 ($\delta = 1 \rightarrow 10$)

$$\therefore \frac{I_E\, s}{h} = .3020$$

$$\therefore I_E = \frac{.3020\, h}{s} = \frac{.3020\,(13')}{17'} = .231$$

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THEN $FS_{PIPING} = \frac{I_{CR}}{I_E}$

WHERE I_{CR} = CRITICAL GRADIENT @ WHICH PIPING OCCURS

$$I_{CR} = \frac{G_s - 1}{1+e}$$ (SEE ATTACHED APP. A)

WHERE G_s = SPECIFIC GRAVITY OF SOIL @ BASE OF EXCAV.

e = VOID RATIO OF SOIL @ BASE OF EXCAV.

FROM DISCUSSION IN APP. A:

$I_{CR} \approx 1.0$ FOR TYPICAL G_s + e VALUES FOR GRANULAR SOILS

$\therefore FS_{PIPING}(d=3') = 1.0/.179 = 5.59$

$FS_{PIPING}(d=5') = 1.0/.231 = 4.33$

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AS NOTED ON PG. A-4 OF APPENDIX A, A MIN. FS_{PIPING} OF 4 TO 5 IS DESIRABLE.

$\therefore$ $FS_{PIPING}(d = 3') \rightsquigarrow$ OK

$FS_{PIPING}(d = 5') \rightsquigarrow$ OK BUT MARGINAL

$\therefore$ PIPING FAILURE OF THE SOILS AT THE BASE OF THE EXCAVATED AREA NEAR THE SHEET PILING DUE TO THE UPWARD FLOW GRADIENT GENERATED FROM THE DIFFERENTIAL HEAD BETWEEN THE WATER SURFACE IN THE STREAM & THE DEWATERED EXCAVATED AREA SHOULD NOT OCCUR.

W. L. Deutsch, Ph.D., P.E.

3/5/02

APPENDIX A

A-1

($\Delta n/\Delta s = 1$ is most sensitive to visual inspection) which reduce to perfect squares in the limit as the number of lines is increased, then one has obtained an unique solution of Laplace's equation for the flow region from which the quantity of seepage, seepage pressures, etc., can be had easily. For example, designating N_f as the number of flow channels and N_e as the number of equipotential drops along each of the channels, we have immediately from Eq. (3) (with $\Delta n/\Delta s = 1$) for the quantity of seepage

$$q = N_f k \,\Delta \bar{q} = \frac{N_f}{N_e} kh \tag{4}$$

where $h = N_e \,\Delta h$ is the total loss in head. In Fig. 1-15 we see that N_f equals about 5 and N_e equals 16.

The following procedure is suggested for the construction of a flow net:

1. Draw the boundaries of the flow region to scale so that all equipotential lines and streamlines that are drawn can be terminated on these boundaries.
2. Sketch lightly three or four streamlines, keeping in mind that they are only a few of the infinite number of curves that must provide a smooth transition between the boundary streamlines. As an aid in the spacing of these lines, it should be noted that the distance between adjacent streamlines increases in the direction of the larger radius of curvature.
3. Sketch the equipotential lines, bearing in mind that they must intersect all streamlines, including the boundary streamlines, at right angles and that the enclosed figures must be squares.*
4. Adjust the locations of the streamlines and the equipotential lines to satisfy the requirements of step 3. This is a trial-and-error process with the amount of correction being dependent upon the position of the initial streamlines. The speed with which a successful flow net can be drawn is highly contingent on the experience and judgement of the individual. In this regard, the beginner will find the suggestions in A. Casagrande's paper [14] to be of particular assistance.
5. As a final check on the accuracy of the flow net, draw the diagonals of the squares. These should also form smooth curves which intersect each other at right angles.

1-13. Seepage Force and Critical Gradient

By virtue of the viscous friction exerted on water flowing through the soil pores, an energy transfer is effected between the water and the soil. The measure of this transfer we found to be the head loss (Δh of Fig. 1-5) between the points under consideration (Δs). The force corresponding to this energy transfer is called the *seepage force*. It is this seepage force

* See previous footnote.

A-3

Taking the gradient of both sides of this equation we obtain

$$\frac{1}{\gamma_w} \operatorname{grad} p = \operatorname{grad} h - \mathbf{j} \tag{6}$$

where $\mathbf{j}$ is a unit vector, as before. Multiplying Eq. (6) by γ_w and replacing grad h by $-i$, the hydraulic gradient, we have the vector equation

$$\operatorname{grad} p = -i\gamma_w - \mathbf{j}\gamma_w \tag{7}$$

Equation (7) is plotted as triangle $OO'M$ in Fig. 1-16.* $i\gamma_w(OM)$ represents the seepage force per unit volume, the direction of which is normal to the equipotentials; $R(O''M)$ represents the magnitude and direction of the resultant force (per unit volume) acting within the pore water at a point in the soil.

For $R = 0$, we see immediately from Fig. 1-16 that a quick condition is incipient if

$$i_{cr} = \frac{S_s - 1}{1 + e} = \frac{\gamma_m'}{\gamma_w} \tag{8}$$

Substituting typical values of $S_s = 2.65$ (quartz sand) and $e = 0.65$ (for sand, $0.57 \leqq e \leqq 0.95$) we see that as an average value the critical gradient can be taken as

$$i_{cr} \approx 1 \tag{9}$$

When information is lacking as to the specific gravity and void ratio of the soil, the critical gradient is generally taken as unity [Eq. (9)].

Equations (8) and (9) provide the basis for stability determinations of the factor of safety against a quick condition (called *piping*). In essence the procedure requires the determination of the maximum hydraulic gradient along the discharge boundary, called the *exit gradient*, which will yield the minimum resultant force (R_{min}) at this boundary. This can be done analytically, as will be demonstrated later, or graphically from flow nets, after a method by Harza [54]. In the graphical method, the gradients along the discharge boundary are taken as the macrogradient across the contiguous squares of the flow net. As the gradients along this boundary vary inversely with the distance between adjacent equipotential lines, it is evident that the maximum exit gradient is located where the vertical projection of this distance is a minimum, such as at the toe of the dam (point C) in Fig. 1-15. For example, the head lost in the final square of Fig. 1-15 is one-sixteenth of the total head loss of 16 ft, or 1 ft, and, as this loss occurs in a vertical distance of approximately 4 ft, the exit gradient at point C is approximately 0.25. Once the magnitude of the exit gradient has been found, the factor of safety with respect to piping is then ascertained by comparing this gradient with the critical gradient

* This is Risenkampf's *triangle of filtration* [122].

A-4

of Eqs. (8) or (9). For example, the factor of safety with respect to piping for the flow condition of Fig. 1-15 is 1.0/0.25 or 4.0. Factors of safety of 4 to 5 are generally considered reasonable for the graphical method of analysis.

1-14. Anisotropy

If the coefficient of permeability is independent of the direction of the velocity, the soil is said to be an *isotropic* flow medium. Moreover, if the soil has the same coefficient of permeability at all points within the region of flow, the soil is said to be *homogeneous* and *isotropic*. If the coefficient of permeability is dependent on the direction of the velocity and if this directional dependence is the same at all points of the flow region, the soil is said to be homogeneous and *anisotropic*. In homogeneous and anisotropic soils the coefficient of permeability is dependent on the direction of the velocity but independent of the space coordinates.

Most soils are anisotropic to some degree. Sedimentary soils often exhibit thin alternating layers. Stratification may result from particle orientation. Generally, in homogeneous natural deposits, the coefficient of permeability in the horizontal direction is greater than that in the vertical. One exception, worthy of special note, is loess, where, because of the vertical structure, the opposite is true.

Although Darcy's law was obtained initially from considerations of one-dimensional macroscopic flow only, in Sec. 1-9, upon the introduction of the velocity potential ϕ, it was demonstrated that the vectorial generalization of Darcy's law was valid for an isotropic flow medium. To provide a theoretical framework for any flow system it is necessary that this generalization take into account the directional dependence of the coefficient of permeability. Thus, it is generally assumed that

$$\mathbf{v}_n = -k_n \operatorname{grad}_n h \tag{1}$$

where k_n is the coefficient of permeability in the n direction and $\mathbf{v}_n$ and $\operatorname{grad}_n h$ are the components of the velocity and the hydraulic gradient in the same direction. For two-dimensional flow in the xy plane the velocity components in the x and y direction are

$$\begin{aligned} u &= -k_x \operatorname{grad}_x h = -k_x \frac{\partial h}{\partial x} \\ v &= -k_y \operatorname{grad}_y h = -k_y \frac{\partial h}{\partial y} \end{aligned} \tag{2}$$

The work of this section will be divided into four parts: (1) It will be shown that a stratified medium of thin homogeneous and isotropic layers can be converted into an equivalent single homogeneous and isotropic layer. (2) It will be shown that the square root of the direc-